SURVEYING MANUAL.

PENCE AND KETCHUM.

Compliments of the Author

A MANUAL OF

FIELD AND OFFICE METHODS

FOR THE USE OF

STUDENTS IN SURVEYING.

BY

WILLIAM D. PENCE,
Professor of Civil Engineering,
Purdue University.

AND

MILO S. KETCHUM,
Assistant Professor of Civil Engineering,
University of Illinois.

PUBLISHED BY THE AUTHORS.

MORNING JOURNAL PRESS.
LAFAYETTE, IND.
1900.

TABLE OF CONTENTS.

TABLE OF CONTENTS.

TABLE OF CONTENTS.

TABLE OF CONTENTS.

PREFACE.

In preparing this manual the following points have been kept especially in view: (1) To provide a simple and comprehensive text designed to anticipate and supplement, rather than replace, the usual elaborate treatise. (2) To bring the student into immediate familiarity with approved surveying methods. (3) To cultivate the student's skill in the rare arts of keeping good field notes and making reliable calculations.

It is believed that the discussions of the different instruments, their use and theory, at the beginning of the several chapters is unusually simple, especially in the relations of the elementary lines.

The several series of practice problems at the conclusion of the respective chapters are arranged so as to give the student familiarity with the use of the instrument before taking up its theory and adjustments, this order being more effective than the reverse. The interest of the student may be stimulated and his gain in skill promoted by giving him practice with level and transit very early in the course, after which the scope of the work may be much more flexible both for student and instructor.

Since the list of problems is more extended than can be covered in the time usually available for surveying field practice, some range is permitted in the choice of work from year to year and under varying local conditions. By using some discrimination in selecting the more important problems for actual field work, the others may be covered sufficiently by class room discussions.

The consistent treatment of errors of surveying receives attention throughout the book. The methods of work both in field and office are designed both to reveal and, as far as possible, to eliminate blunders and errors, and the tests of precision are borrowed from the most rational current practice. The distribution of residual errors falling within the permissible limits likewise receives due consideration.

An important innovation in this manual is the liberal use of field note and other forms executed according to the standard required of the student in like work. The high value of such samples in developing the student's skill in this important detail of field work has been well established. It will be seen that the forms are prescribed in liberal number in the earlier stages of the work while the student is engaged in fixing a standard of quality, but that farther on he is required more and more to devise his own forms. A valuable feature of this system is the liberal amount of practice obtained in freehand lettering and the marked effect on the drafting and other kinds of work.

It is suggested that the student should be trained to be self-reliant by requiring him to verify his own results before submitting them for criticism. Likewise he should be encouraged to be genuine by placing him on his honor.

This somewhat informal guide to field and office methods is issued primarily for the use of the authors' classes, but it is hoped that others as well may find it of value in presenting principles to the beginner, and in cultivating his spirit and manual skill.

December, 1900

W. D. P.
M. S. K.

SPECIFICATIONS FOR A GOOD ENGINEER.

"A good engineer must be of inflexible integrity, sober, truthful, accurate, resolute, discreet, of cool and sound judgment, must have command of his temper, must have courage to resist and repel attempts at intimidation, a firmness that is proof against solicitation, flattery or improper bias of any kind, must take an interest in his work, must be energetic, quick to decide, prompt to act, must be fair and impartial as a judge on the bench, must have experience in his work and in dealing with men, which implies some maturity of years, must have business habits and knowledge of accounts. Men who combine these qualities are not to be picked up every day. Still they can be found. But they are greatly in demand, and when found, they are worth their price; rather they are beyond price, and their value can not be estimated by dollars."—*Chief Engineer Starling's Report to the Mississippi Levee Commissioners.*

CHAPTER I.

GENERAL INSTRUCTIONS.

FIELD WORK.

Habitual Correctness.—Habitual correctness is a duty. Error should be looked upon as *probable*, and every precaution taken to verify data and results. Unchecked work may always be regarded as doubtful. A discrepancy which is found by the maker in time to be corrected by him before any damage is done is not necessarily discreditable, provided the error is not repeated. However, *habitual error* is not only discreditable but dishonorable as well, and nothing except intentional dishonesty injures the reputation of the engineer more quickly or permanently.

Consistent Accuracy.—The degree of precision sought in the field measurements should be governed strictly by the dictates of common sense and experience. Due consideration of the purposes of the survey and of the time available will enable one to avoid extreme precision when ordinary care would suffice, or crudeness when exactness is required, or inconsistency between the degrees of precision observed in the several parts of the survey. It is a very common practice of beginners, and of many experienced engineers as well, to carry calculated results far beyond the consistent exactness.

Speed.—Cultivate the habit of doing the field work quickly as well as accurately. True skill involves both quantity and quality of results. However, while the habit of rapid work can and should be acquired, the speed attempted in any given problem should never be such as to cast doubt upon the results. Slowness due to laziness is intolerable.

Familiarity with Instructions.—The instructions for the day's work should be read over carefully, and preliminary steps, such as the preparation of field note forms, should be taken so as to save time and make the work in the field as effective as possible. The ability and also the desire to understand and obey instructions are as essential as the skill to execute them.

Inferior Instruments.—Should a poor instrument or other equipment be assigned, a special effort should be made to secure excellent results. In actual practice, beginners often have to work with defective instruments, but they should never seek, nor are they permitted, to justify poor results by the character of the field equipment. The student should therefore welcome an occasional opportunity to secure practice with poor instruments.

Alternation of Duties.—The members of each party should alternate in discharging the several kinds of service involved in the field problems, unless otherwise instructed. Training in the subordinate positions is essential whether the beginner is to occupy them in actual practice or not, for intelligent direction of work demands thorough knowledge of all its details.

Field Practice Decorum.—The decorum of surveying field practice should conform reasonably to that observed in other laboratory work.

THE CARE OF FIELD EQUIPMENT.

Responsibility.—The student is responsible for the proper use and safe return of all equipment. All cases of breakage, damage, loss or misplacement must be reported promptly. The equipment should be examined when assigned and an immediate report made of any injury or deficiency, so that responsibility may be properly fixed.

PRECAUTIONS.—Careful attention to the following practical suggestions will save needless wear to the equipment and reduce the danger of accidents to a minimum, besides adding to the quality and speed of the work.

Tripod.—Inspect the tripod legs and shoes. The leg is of the proper tightness, if when lifted to an elevated position it sinks gradually of its own weight. The tripod shoes should be tight and have reasonably sharp points.

Setting Up Indoors.—In setting up the instrument indoors press the tripod shoes firmly into the floor, preferably with each point in a crack. Avoid disturbing other instruments in the room.

Instrument Case.—Handle the instrument gently in removing it from and returning it to the case. It is always best to place the hands beneath the leveling base in handling the detached instrument. Considerable patience is sometimes required to close the lid after returning the instrument.

Mounting the Instrument.—See that the instrument is securely attached to the tripod before shouldering it. Undue haste in this particular sometimes results in costly accidents. When screwing the instrument on the tripod head, it should be turned in a reverse direction until a slight jar is felt, indicating that the threads are properly engaged.

Sunshade.—Always attach the sunshade regardless of the kind of weather. The sunshade is a part of the telescope tube and the adjustment of a delicate instrument may sometimes be affected by its absence. In attaching or removing the sunshade or object glass cap, always hold the telescope tube firmly with one hand and with the other twist the shade or cap *to the right* to avoid unscrewing the object glass cell.

Carrying the Instrument.—Do not carry the instrument on the shoulder in passing through doors or in climbing fences. Before shouldering the instrument, the principal motions should be slightly clamped; with the transit, clamp the telescope on the line of centers; and with the level, when the telescope is hanging down. In passing through timber with low branches, give special attention to the instrument. Before climbing a fence, set the instrument on the opposite side with tripod legs well spread.

Setting Up in the Field.—When setting up in the field, bring the tripod legs to a firm bearing with the plates approximately level. Give the tripod legs additional spread in windy weather or in places where the instrument may be subject to vibration or other disturbance. On side-hill work place one leg up hill. With the level, place two tripod shoes on the general direction of the line of levels.

Exposure of Instrument.—Do not expose the instrument to rain or dampness. In threatening weather the waterproof bag should be taken to the field. Should the instrument get wet, wipe it thoroughly dry before returning it to the case. Protect the instrument from dust and dirt, and avoid undue exposure to the burning action of the sun. Avoid subjecting it to sudden changes of temperature. In cold weather when bringing an instrument indoors cover the instrument with the bag or return it to the case immediately to protect the lenses and graduations from condensed moisture.

Guarding the Instrument.—Never leave an instrument unguarded in exposed situations, such as in pastures, near driveways, or where blasting is in progress. Never leave an instrument standing on its tripod over night in a room.

Manipulation of Instrument.—Cultivate from the very beginning the habit of delicate manipulation of the instrument. Many parts, when once impaired, can never be restored to their original condition. Rough and careless treatment of field instruments is characteristic of the unskilled observer. Should any screw or other part of the instrument work harshly, call immediate attention to it so that repairs may be made. Delay in such matters is very destructive to the instrument.

Foot Screws.—In leveling the instrument, the foot screws should be brought just to a snug bearing. If the screws are too loose, the instrument rocks, and accurate work can not be done; if too tight, the instrument is damaged, and the delicacy and accuracy of the observations are reduced. Much needless wear of the foot screws may be avoided if the plates are brought about level when the instrument is set up. With the level, a pair of foot screws should be shifted to the general direction of the back or fore sight before leveling up.

Eyepiece.—Before beginning the observations, focus the eyepiece perfectly on the cross-hairs. This is best done by holding the note book page, handkerchief, or other white object a foot or so in front of the object glass so as to illuminate the hairs; and then, by means of the eyepiece slide, focus the microscope on a speck of dust on the cross-hairs near the middle of the field. To have the focusing true for natural vision, the eye should be momentarily closed several times between observations in order to allow the lenses of the eye to assume their normal condition. The omission of this precaution strains the eye and is quite certain to cause parallax. After the eyepiece is focused on the cross-hairs, test for parallax by sighting at a well defined object and observing whether the cross-hairs seem to move as the eye is shifted slightly.

Clamps.—Do not overstrain the clamps. In a well designed instrument the ears of the clamp screw are purposely made small to prevent such abuse. Find by experiment just how tight to clamp the instrument in order to prevent slipping, and then clamp accordingly.

Tangent Screws.—Use the tangent screws only for slight motions. To secure even wear the screws should be used equally in all parts of their length. The use of the wrong tangent movement is a fruitful source of error with beginners.

Adjusting Screws.—Unless the instrument is assigned expressly for adjustment, do not disturb the adjusting screws.

Magnetic Needle.—Always lift the needle before shouldering the instrument. Do not permit tampering with the needle. If possible, avoid subjecting the needle to magnetic influences, such as may exist on a trolley car. Should the needle become reversed in its polarity or require remagnetization, it may be removed from the instrument and brought into the magnetic field of a dynamo or electric motor for several minutes, the needle being jarred slightly during the exposure; or a good bar or horshoe magnet may be used for the same purpose. The wire coil counterbalance on the needle will usually require shifting after the foregoing process.

Lenses.—Do not remove or rub the lenses of the telescope. Should it be *absolutely necessary* to clean a lens, use a very soft rag with caution to avoid scratching or marring the polished surface. Protect the lenses from flying sand

and dust, which in time seriously affect the definition of the telescope.

Plumb Bob.—Do not abuse the point of the plumb bob and avoid needless knots in the plumb bob string.

Cleaning Tripod Shoes.—Remove the surplus soil from the tripod shoes before bringing the instrument indoors.

Leveling Rods.—Leveling rods and stadia boards should not be leaned against trees or placed where they may fall. Avoid injury to the clamps, target and graduations. Do not mark the graduations with pencil or otherwise. Avoid needless exposure of the rod to moisture or to the sun.

Flag Poles.—Flag poles should not be unduly strained, and their points should be properly protected.

Chains and Tapes.—Chains should not be jerked. Avoid kinks in steel tapes, especially during cool weather. When near driveways, in crowded streets, etc., use special care to protect the tape. Band tapes will be done up in 5-foot loops, figure 8 form, unless reels are provided. Etched tapes should be wiped clean and dry at the end of the day's work.

Axes and Hatchets.—Axes and hatchets will be employed for their legitimate purposes only. Their wanton use in clearing survey lines is forbidden, and their use at all for such purpose on private premises must be governed *strictly* by the rights of the owner.

Stakes.—The consumption of stakes should be controlled by reasonable economy. Surplus stakes will be returned to the general store. For the protection of mowing machines in meadows, etc., hub stakes should be driven flush with the surface of the ground, and other stakes should be left high enough to be visible. Whenever practicable, stakes which may endanger machines should be removed after serving the purpose for which they were set.

FIELD NOTES.

Scope of Field Notes.—The notes should be a complete record of each day's work in the field. In addition to the title of the problem and the record of the data observed, the field notes should include the date, weather, organization of party, equipment used, time devoted to the problem, and any other information which is at all likely to be of service in connection with the problem. No item properly belonging to the notes should be trusted to memory. Should the question arise as to the desirability of any item, it is always safe to include it. The habit of rigid self criticism of the field notes should be cultivated.

Character of Notes.—The field notes should have character and force. As a rule, the general character of the student's work can be judged with considerable certainty by the appearance of his field notes. A first-class page of field notes always commands respect, and tends to establish and stimulate confidence in the recorder. The notes should be arranged systematically.

Interpretation of Notes.—The field notes should have one and only one reasonable interpretation, and that the correct one. They should be perfectly legible and easily understood by anyone at all familiar with such matters.

Original Notes.—Each student must keep complete notes of each problem. Field notes must not be taken on loose slips or sheets of paper or in other note books, but the *original record* must be put in the prescribed field note book *during the progress of the field work.*

Field Note Book.—The field record must be kept in the prescribed field note book. For ease of identification the name of the owner will be printed in bold letters at the top of the front cover of the field note book.

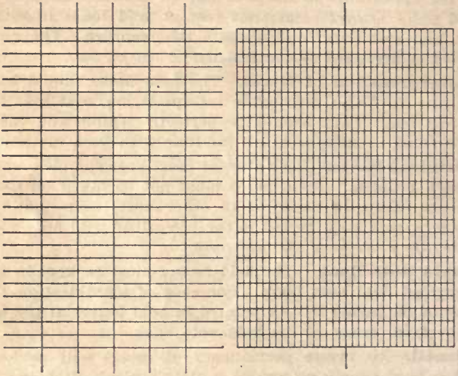
Pencil.—To insure permanency all notes will be kept with a hard pencil, preferably a 4H. The pencil should be kept well sharpened and used with sufficient pressure to indent the surface of the paper somewhat.

Title Page.—An appropriate title page will be printed on the first page of the field note book.

Indexing and Cross Referencing.—A systematic index of the field notes will be kept on the four pages following the title page. Related notes on different pages will be liberally and plainly cross referenced. The pages of the note book will be numbered to facilitate indexing.

Methods of Recording Field Notes.—There are three general methods of recording field notes, namely, (1) by sketch, (2) by description or narration, and (3) by tabulation. It is not uncommon to combine two or perhaps all three of these methods in the same problem or survey.

Form of Notes.—All field notes must be recorded in the form below, except where circumstances require modification. If no form is given, the student will devise one suited to the needs of the particular problem.



Lettering.—Field notes will be printed habitually in the "Engineering News" style of freehand lettering, as treated in Reinhardt's "Freehand Lettering." The body of the field notes will be recorded in the slanting letter and the headings will be made in the upright letter. The former slants to the right 1:2.5 and the so-called upright letter is made to slant to the left slightly, say 1:25. Lower case letters will be used in general, capitals being employed for initials and important words, as required. In the standard field note alphabet the height of lower case letters a, c, e, i, m, n, etc., is 3-50 (say 1-16) inch, and the height of lower case b, d, f, g, h, etc., and of all capital letters and all numerals is 5-50 (1-10) inch; lower case t is made four units (4-50) inch high. This standard accords with best current practice and is based upon correct economic principles. (See chapter giving discussion of freehand lettering.) The standard field note alphabets are given on the bookmark scale which accompanies this manual. The student is expected to make the most of this opportunity to secure a liberal amount of practice in freehand lettering.

Field Note Sketches.—Sketches will be used liberally in the notes and will be made *in the field.* If desired, a ruler may be used in drawing straight lines, but the student is urged to acquire skill at once in making good plain freehand sketches. The field sketches should be bold and clear, in fair proportion, and of liberal size so as to avoid confusion of detail. The exaggeration of certain details in a separate sketch sometimes adds greatly to the clearness of the notes. The sketches should be supplemented by descriptive statements when helpful, and important points of the sketch should be lettered for reference. The precise scaling of sketches in the field note book, while sometimes necessary, is usually undesirable owing to the time consumed. It is also found that undue attention to the drafting of the sketch is very apt to occupy the mind and cause omissions of important numerical data. Since recorded figures and not the size of the field sketch itself must usually be employed in the subsequent use of the notes, it is important to review the record *before leaving the field* to detect omissions or inconsistencies. Making sketches on loose sheets or in other books and subsequently copying them into the regular field book is very objectionable practice and will not be permitted in the class work. Copies of field notes or sketches are never as trustworthy as the original record made *during the progress* of the field work. In very rapid surveys where legibility of the original record must perhaps suffer somewhat, it is excellent practice to transcribe the notes at once to a neighboring page, thus preserving the original rough notes for future reference. The original has more weight as evidence, but the neat copy made before the notes are "cold" is of great help in interpreting them.

Numerical Data.—The record of numerical data should be consistent with the precision of the survey. In observations of the same class a uniform number of decimal places should be recorded. When the fraction in a result is exactly one-half the smallest unit or decimal place to be observed, record the even unit. Careful attention should be given to the *legibility of numerals*. This is a matter in which the beginner is often very weak. This defect can be corrected best by giving studious attention and practice to both the form and vertical alinement of tabulated numerals.

Erasures.—Erasures in the field notes will be strictly avoided. Should a figure be incorrectly recorded, it should be crossed out and the correct entry made near by. The neat cancellation of an item in the notes inspires confidence, but evidence of an erasure or alteration casts doubt upon their genuineness. When a set of notes becomes so confused that erasure seems desirable, it should be transcribed, usually on another page. Rejection of a page of notes should be indicated by a neat cross mark, and cross reference should be made between the two places.

Office Copies.—Office copies of field notes will be submitted promptly, as required. These copies must be actual transcripts from the original record contained in the field note book of the individual submitting the copy. When office copies are made, a memorandum of the fact should be entered on the page of the field note book. When so specified, the office copies will be executed in india ink.

Criticism of Field Notes.—The field notes must be kept in shape for inspection at any time, and be submitted on call. All calculations and reductions must be kept up to date. The points to which chief attention should be directed in the criticism of the field notes are indicated in the following schedule. The student is expected to criticise his own notes and submit them in as perfect condition as possible. For simplicity the criticisms will be indicated by stamping on the note book page the reference letters and numbers shown in the schedule.

SCHEDULE OF POINTS FOR THE CRITICISM OF FIELD NOTE BOOKS.

A. SUBJECT MATTER.

(1) **General:**

(a) Descriptive title of problem.

(b) Date.

(c) Weather.

(d) Organization of party.

(e) Equipment used.

(f) Time devoted to the problem.

(g) Indexing and cross referencing.

(h) Page numbering.

(i) Title page.

(j) Identification of field note book.

(2) **Record of Data:**

(a) Accuracy.

(b) Completeness.

(c) Consistency.

(d) Arrangement.

(e) Originality.

B. EXECUTION.

(1) **Lettering:**

(a) Style. ("Engineering News.")

(b) Size. (a, c, e, i, etc., 3-50 (say 1-16) inch high; b, d, f, g, etc., A, B, C, etc., and 1, 2, 3, etc., 5-50 (1-10) inch high; t, 4-50 inch.)

(c) Slant. (In body of notes, "slanting," 1:2.5 right; in headings, "upright," about 1:25 to left.)

(d) Form. (See Reinhardt's "Freehand Lettering.")

(e) Spacing. (Of letters in words; of numerals; of words; balancing in column or across page.)

(f) Alinement. (Horizontal; vertical.)

(g) Permanency. (Use sharp hard pencil with pressure.)

(2) **Sketches.**

(a) To be bold, clear and neat.

(b) To be ample in amount.

(c) To be of liberal size.

(d) To be in fair proportion.

(e) To be made freehand.

(f) To be made in the field.

Importance of Office Work.—Capable office men are comparatively rare. Skill in drafting and computing is within the reach of most men who will devote proper time and effort to the work. Men who are skillful in both field and office work have the largest opportunity for advancement.

Calculations.—All calculations and reductions of a permanent character must be shown in the field note book in the specified form. Cross references between field data and calculations should be shown. Consistency between the precision of computed results and that of the observed data should be maintained. Computed results should be verified habitually, and the verified results indicated by a check mark. Since most computers are prone to repeat the same error, it is desirable in checking calculations to employ independent methods and to follow a different order. A fruitful source of trouble is in the transcript of data, and this should be checked first when reviewing doubtful calculations. Skilled computers give much attention to methodical arrangement, and to contracted methods of computing and verifying results. Familiarity with the slide rule and other labor saving devices is important. (See chapter on methods of computing.)

Drafting Room Equipment.—The student is responsible for the proper use and care of drafting room furniture and equipment provided for his use.

Drafting.—The standard of drafting is that indicated in Reinhardt's "Technic of Mechanical Drafting."

Drafting Room Decorum.—The decorum of the student in the drafting room will conform to that observed in first-class city drafting offices.

———o———

CHAPTER II.

THE CHAIN AND TAPE.

METHODS OF FIELD WORK.

Units of Measure.—In the United States the foot is used by civil engineers in field measurements. Fractions of a foot are expressed decimally, the nearest 0.1 being taken in ordinary surveys, and the nearest 0.01 foot (say 1-8 inch) in more refined work.

In railroad and similar "line" surveys in which a station stake is set every 100 feet, the unit of measure is really 100 feet instead of the foot. The term "station" was originally applied only to the actual point indicated by the numbered stake, but it is now universal practice in this country to use the word station in referring to either the point or the 100-foot unit distance. A fractional station is called a "plus" for the reason that a plus sign is used to mark the decimal point for the 100-foot unit, the common decimal point being reserved for fractions of a foot. The initial or starting stake of such a survey is numbered 0.

The 100-foot chain is commonly called the "engineers' chain" to distinguish it from the 66-foot or 100-link chain which is termed the "surveyors' chain" because of its special value in land surveys involving acreage. The latter is also called the Gunter chain after its inventor, and is otherwise known as the four-rod or four-pole chain. British engineers use the Gunter chain for both line and land surveys. The United States rectangular surveys were made throughout with the 66-foot chain.

In the Spanish-American countries the vara is generally used in land surveys. The Castilian vara is 32.8748 inches long, but the state of California has adopted 32.372 inches, and Texas 33 1-3 inches, as the legal length of the vara.

While the metric system is used exclusively or in part in each of the several United States government surveys, except the public land surveys, little or no progress has been made toward its introduction in other than government surveys.

Linear Measuring Instruments.—Two general types of linear measuring devices are used by surveyors, viz., the common chain and the tape. There are several kinds of each, according to the length, material and method of graduation.

Fig. 1.

The common chain is made up of a series of links of wire having loops at the ends and connected by rings so as to afford flexibility. The engineers' chain is shown in (a), Fig. 1, the illustration being that of a 50-foot chain, or one-half the length generally used. The surveyors' or Gunter chain is shown in (b), Fig. 1. In the common chain the end graduation is the center of the cross bar of the handle, and every tenth foot or link is marked by a notched brass tag. In the 100-foot or 100-link chain the number of points on the tag indicates the multiple of ten units from the nearer end, and a circular tag marks the middle of the chain. The chain is done up hour glass shape, as shown in the cut.

Chaining pins made of steel wire are used in marking the end of the chain or tape in the usual process of linear measurement. A set of pins usually numbers eleven, as indicated at (c), Fig. 1. The pins are carried on a ring made of spring steel wire.

The flat steel band, shown in (d) and (e), Fig. 1, is the best form of measuring device for most kinds of work. The band tape is usually 100 feet long. The end graduations of the band tape are usually indicated by brass shoulders, which should point in the same direction, as shown in (f), Fig. 1. The 100-foot band tape is commonly graduated every foot of its length, and the end foot to every 0.1 foot, every fifth foot being numbered on a brass sleeve. Brass rivets are the most common mode of graduating this tape. The band tape may be rolled up on a special reel, as indicated in (d) and (e), although some engineers dispense with the reel and do up the tape in the form of the figure 8 in loops of five feet or so.

The steel tapes shown in (g) and (h) have etched graduations. This style of tape is commonly graduated to 0.01 foot or 1-8 inch. It is more fragile than the band tape and is commonly used on more refined work. The form of the case shown in (h) has the advantage of allowing the tape to dry if wound up while damp.

The "metallic" tape, (i), Fig. 1, is a woven linen line having fine brass wire in the warp.

The steel tape is superior to the common chain chiefly because of the permanency of its length. The smoothness and lightness of the steel tape are often important advantages, although the latter feature may be a serious drawback at times. The tape is both easier to break and more difficult to mend than the common chain.

Chaining.—In general, the horizontal distance is chained. Two persons, called head and rear chainmen, are required. The usual process is as follows:

The line to be chained is first marked with range poles. The head chainman casts the chain out to the rear, and after setting one marking pin at the starting point and checking up the remaining ten pins on his ring, steps briskly to the front. The rear chainman allows the chain to pass through his hands to detect kinks and bent links. Just before the full length is drawn out, the rear chainman calls "halt," at which the head chainman turns, shakes out the chain and straightens it on the true line under the direction of the rear chainman. In order to allow a clear sight ahead, the front chainman should hold the chain handle with a pin in his right hand well away from his body, suporting the right elbow on the right knee, if desired. The rear chainman holds the handle in his left hand approximately at the starting point and motions with his right to the head chainman, his signals being distinct both as to direction and amount. Finally, when the straight and taut chain has been brought practically into the true line, the rear chainman, slipping the handle behind the pin at the starting point with his left hand, and steadying the top of the pin with his right, calls out "stick." The head chainman at this instant sets his pin in front of the chain handle and responds "stuck," at which signal *and not before* the rear chainman pulls the pin.

Both now proceed, the rear chainman giving the preliminary "halt" signal as he approaches the pin just set by the head chainman. The chain is lined up, stretched, the front pin set, and the rear pin pulled on signal, as described for the first chain length. This process is repeated until the head chainman has set his tenth pin, when he calls "out" or "tally," at which the rear chainman walks ahead, counting his pins as he goes and, if there are ten, transfers them to the head chainman who also checks them up and replaces them on his ring. A similar check in the pins may be made at any time by remembering that the sum, omitting the one in the ground, should be ten. This safeguard should be taken often to detect loss of pins. The count of tallies should be carefully kept.

When the end of the line is reached, the rear chainman steps ahead, and reads the fraction at the pin, noting the units with respect to the brass tags on the chain. The number of pins in the hand of the rear chainman indicates the number of applications of the chain since the starting or last tally point. A like method is used in case intermediate points are to be noted along the line.

On sloping ground the horizontal distance may be obtained either by leveling the chain and plumbing down from the elevated end, or by measuring on the slope and correcting for the inclination. In ordinary work the former is preferred, owing to its simplicity. In "breaking chain" up or down a steep slope, the head chainman first carries the full chain ahead and places it carefully on the true line. A plumb bob, range pole or loaded chaining pin should be used in plumbing the points up or down. The segments of the chain should be in multiples of ten units, as a rule, and the breaking points should be "thumbed" by both chainmen to avoid blunders. Likewise, special caution is required to avoid confusion in the count of pins during this process.

The general method of measuring with the band tape is much the same as with the common chain. The chief difference is due to the fact that the handle of the tape extends beyond the end graduation, so that it is more convenient for the head chainman to hold the handle in his left hand and rest his left elbow on his left knee, setting the pin with his right hand. Another difference is in the method of reading fractions. It is best to read the fraction *first by estimation*, as with the chain, making sure of the *feet*; then shifting the tape along one foot, getting an exact decimal record of the fraction by means of the end foot graduated to tenths; the nearest 0.01 foot is estimated, or in especially refined work, read by scale.

In railroad and similar line surveys, chaining pins are usually dispensed with and the ends of the chain are indicated by numbered stakes. The stake marked 0 corresponds to the pin at the starting point, and the station stakes are marked thence according to the number of 100-foot units laid off.

Perpendiculars.—Perpendiculars may be erected and let fall with the chain or tape by the following methods:

(a) By the 3:4:5 method, shown in (a), Fig. 2, in which a triangle having sides in the ratio stated, is constructed.

(b) By the chord bisection method, shown in (b), Fig. 2, in which a line is passed from the bisecting point of the chord to the center of the circle, or vice versa.

(c) By the semicircle method, shown in (c), Fig. 2, in which a semicircle is made to contain the required perpendicular.

The first method corresponds to the use of the triangle in drafting. Good intersections are essential in the second and third methods. Results may be verified either by using another process, or by repeating the same method with the measurements or position reversed, as indicated in (d), Fig. 2.

In locating a perpendicular from a remote point, the ratio method shown in (e), Fig. 2, may be used; or a careful trial perpendicular may be erected at a point estimated by placing the heels squarely on line and swinging the arms to the front, then proving by precise method.

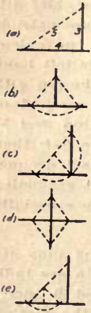


Fig. 2.

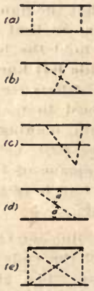


Fig. 3.

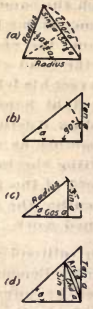


Fig. 4.

Parallels.—Parallels may be laid off with the chain in various ways, a few of the simpler of which are:

(a) By equal distances, as in (a), Fig. 3, in which two equal distances are laid off, usually at right angles to the given line.

(b) By similar triangles, as in (b) and (c), Fig. 3. The ratio may, of course, have any value.

(c) By alternate angles, as in (d), Fig. 3, in which two equal angles are laid off in alternation.

The first method is adapted to laying off a rectangle, as in staking out a building, in which case a good check is found in the equality of the diagonals. Precision of alinement is important, especially where a line is prolonged.

Angles.—Angles may be determined by linear measurements in the following ways:

(a) By the chord method, shown in (a), Fig. 4, in which the radius is laid off on the two lines forming the angle, and the chord measured.

(b) The tangent method, shown in (b), Fig. 4, in which a perpendicular is erected at one end of the radius, and the length of the perpendicular intercepted by the two lines measured.

(c) The sine-cosine method, (c), Fig. 4, which is better suited to constructing than to measuring angles.

The chord method is usually the most satisfactory. The tangent method may be applied to the bisected angle when its value approaches a right angle. Measurement of the supplementary angle affords an excellent check. A 100-foot radius is commonly used, although good results may be had with the 50-foot tape. Careful alinement is of the first importance in angular measurements.

It is sometimes necessary to determine angles, at least approximately, when no tables are at hand. Fair results may be had on smooth ground by measuring the actual arc struck off to a radius of 57.3 feet.

For very small angles, the sine, chord, arc and tangent, (d), Fig. 4, are practically equal. Thus, sin 1° is .017452 and tan 1°, .017455, or either (say) .01745, or 1¾ per cent. Also, arc 1′ is .000291, or (say) .0003 (three zeros three); and, arc 1″ is .00000485, (say) .000005 (five zeros five).

Location of Points.—Points are located in surveying field practice in the following seven ways.

(a) By rectangular coordinates, that is, by measuring the perpendicular distance from the required point to a given line, and the distance thence along the line to a given point, as in (a), Fig. 5.

(b) By focal coordinates or tie lines, that is, by measuring the distances from the required point to two given points, as in (b), Fig. 5.

(c) By polar coordinates, that is, by measuring the angle between a given line and a line drawn from any given point of it to the required point; and also the length of this latter line, as in (c), Fig. 5.

(d) By modified polar coordinates, that is, by a distance from one known point and a direction from another, as in (d), Fig. 5.

(e) By angular intersection, that is, by measuring the angles made with a given line by two other lines starting from given points upon it, and passing through the required point, as in (e), Fig. 5.

(f) By resection, that is, by measuring the angles made with each other by three lines of sight passing from the required point to three points, whose positions are known, as in (f), Fig. 5.

(g) By diagonal intersection, that is, by two lines joining two pairs of points so as to intersect in the required point, as in (g), Fig. 5.

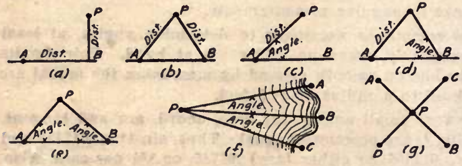


Fig. 5.

In each of these methods, except (f), the point is determined by the intersection of either two right lines, or two circles, or a right line and a circle.

Methods (a) and (b) are best suited to chain surveys; (c) and (d) are used most in the location of railroad curves; (e) and (f) are employed chiefly in river and marine surveys for the location of soundings, the latter being commonly known as the "three-point problem;" the last method, (g), is much used for "referencing out" transit points in railroad and similar construction surveys.

Location of Objects.—The location of buildings and topographic objects usually involves one or more of the foregoing methods of locating a point.

In Fig. 6, (a), (b), (c), and (d) suggest methods of locating a simple form, and (e) and (f) illustrate more complex cases.

Tie Line Surveys.—For many purposes tie line surveys, made with the chain or tape alone, are very satisfactory. The skeleton of such surveys is usually the triangle, the detail being filled in by the methods just outlined. Much time may be saved by carefully planning the survey. A few typical applications of the tie line method are shown in Fig. 7.

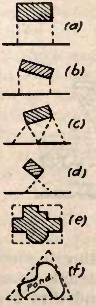


Fig. 6.

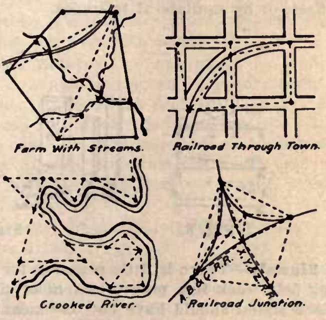


Fig. 7.

Ranging in Lines.—The range or flag pole is usually painted with alternate feet red and white, and the lower end is shod or spiked. A temporary form of range pole, called a picket, is sometimes cut from straight sapplings.

In flagging a point, the spike of the pole is placed on the tack and the pole plumbed by holding it symmetrically between the tips of the fingers of the two hands, the flagman being squarely behind the pole.

In hilly or timbered country the two land corners or other points between which it is desired to range in a line, are often invisible one from the other. In many cases two intermediate points C′ and D′, (a), Fig. 8, may be found, from which the end points B and A, respectively, are visible; so that after a few successive linings in, each by the other, the true points, C and D, are found.

Otherwise, as shown at (b), Fig. 8, a random line may be run from A towards B. The trial line is chained and marked, the perpendicular from B located, and points interpolated on the true line.

If the desired line is occupied by a hedge or other obstruction, an auxiliary parallel line may be established in the adjacent road or field, after one or two trials, as in (c), Fig. 8.

A line may be prolonged past an obstacle by rectangular offsets or by equilateral triangles.

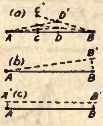


Fig. 8.

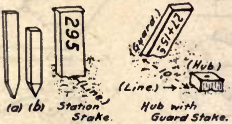


Fig. 9.

Signals.—There is little occasion for shouting in surveying field work if a proper system of sight signals is used. Each signal should have but one meaning and that a perfectly distinct one. Signals indicating motion should at once show clearly both the direction and amount of motion desired. Some of the signals in common use are as follows:

(a) "Right" or "left,"—the arm is extended distinctly in the desired direction and the motion of the forearm and hand is graduated to suit the lateral motion required.

(b) "Up" or "down,"—the arm is extended laterally and raised or lowered distinctly with motions to suit the magnitude of the movement desired. Some levelers use the left arm for the "up" signal and the right for "down."

(c) "Plumb the pole (or rod),"—if to the right, that arm is held vertically with hand extended and the entire body, arm included, is swung distinctly to the right, or vice versa.

(d) "All right,"—both arms are extended full length horizontally and waved vertically.

(e) "Turning point" or "transit point,"—the arm is swung slowly about the head.

(f) "Give line,"—the flagman extends both arms upward, holding the flag pole horizontally, ending with the pole in its vertical position. If a precise or tack point is meant, the signal is made quicker and sharper.

(g) Numerals are usually made by counted vertical swings with the arm extended laterally. A station number is given with the right hand and the plus, if any, with the left; or a rod reading in like manner. The successive counts are separated by a momentary pause, emphasized, if desired, by a slight swing with both hands.

Stakes and Stake Driving.—A flat stake is used to mark the stations in a line survey, and a square stake or hub to mark transit stations, (a) and (b), Fig. 9. The station stake is numbered on the rear face, and the hub is witnessed by a flat guard stake driven slanting 10 inches or so to the left, Fig. 9. The numerals should be bold and distinct, and made with keel or waterproof crayon, pressed into the surface of the wood.

Having located a point approximately with the flag pole, the stake should be driven truly plumb in order that the final point may fall near the center of its top. In driving a stake, the axeman should watch for signals. It is better to draw the stake by a slanting blow than to hammer the stake over after it is driven. Good stake drivers are scarce.

PROBLEMS WITH THE CHAIN AND TAPE.

General Statement.—Each problem is stated under the following heads:

(a) *Equipment.*—In which are specified the articles and instruments assigned or required for the proper performance of the problem. A copy each of this manual and of the regulation field note book, with a hard pencil to keep the record, form part of the equipment for every problem assigned.

(b) *Problem.*—In which the problem is stated in general terms. The special assignments will be made by program.

(c) *Methods.*—In which the methods to be used in the assigned work are described more or less in detail. In some problems alternative methods are suggested, and in others the student is left to devise his own.

PROBLEM A1. LENGTH OF PACE.

(a) *Equipment.*—(No instrumental equipment required.)

(b) *Problem.*—Investigate the length of pace as follows: (1) the natural pace; (2) an assumed pace of 3 feet; and (3) the effect of speed on the length of the pace.

(c) *Methods.*—(1) On an assigned course of known length count the paces while walking at the natural rate. Observe the nearest 0.1 pace in the fraction at the end of the course. Secure ten consecutive results, with no rejections, varying not more than 2 per cent. (2) Repeat (1) for an assumed 3-foot pace. (3) Observe in duplicate time and paces for four or five rates from very slow to very fast, with paces to nearest 0.1 and time to nearest second. Record data and make reductions as in form opposite.

PROBLEM A2. DISTANCES BY PACING.

(a) *Equipment.*—(No instrumental equipment required.)

(b) *Problem.*—Pace the assigned distances.

(c) *Methods.*—(1) Standarize the pace in duplicate on measured base. (2) Pace each line in duplicate, results differing not more than 2 per cent. Record and reduce as in form.

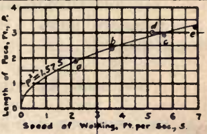
INVESTIGATION OF LENGTH OF PACE.

Sept. 13, 99 (3 hrs.) Clear and cool. J. Doe, Surveyor

Kind of Pace.	No.	Paces per 400 Ft. Paces.	Mean. Paces	Length of Pace. Ft	Remarks. Sept 13, 99 Clear. Smooth ground.
Natural	1	138.0			With the wind
	2	137.4			Against ″
	3	139.0			″ ″
	4	137.6			″ ″
	5	138.0			″ ″
	6	139.0			″ ″
	7	137.5			″ ″
	8	139.0			″ ″
	9	138.0			″ ″
	10	139.3	138.26	2.89	″ ″
		(12.6)			
3-Foot.	1	~~131.0~~			With the wind.
	2	132.6			Against ″
	3	133.0			″ ″
	4	133.4			″ ″
	5	134.0			″ ″
	6	133.3			″ ″
	7	132.0			″ ″
	8	133.0			″ ″
	9	133.3			″ ″
	10	132.6			″ ″
	11	133.0	133.02	3.01	″ ″
		(10.2)			

EFFECT OF SPEED ON LENGTH OF PACE.

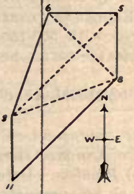
Kind of Pace	Paces in 400 Ft Observed Paces	Mean Paces	Mean Pace, P. Ft.	Time 400 Ft Sec.	Speed of Pacing, S. Ft per Sec.
Very slow	214.6				
″ ″	217.8	216.20	(a) 1.85	182	(a) 2.20
Slow	168.0				
″	167.5	167.75	(b) 2.38	111	(b) 3.60
Natural.	139.4				
″	137.5	138.45	(c) 2.89	71	(c) 5.63
3-Foot.	133.3				
″	133.6	133.45	(d) 3.00	77	(d) 5.20
Fast.	124.7				
″	125.3	125.00	(e) 3.20	58	(e) 6.90



DISTANCES BY PACING.

J Doe, Surveyor. Sept 14, '99 (3 hours) Clear and cool.

Line	No.	Length of Pace Per 400 Ft	Length of Pace Ft	Length of Line Observed Paces	Length of Line Mean Paces	Length of Line Length Ft	
	1	144.0					
	2	142.0					
		143.0	2.80				
5-6	1			134.0			
″	2			134.0	134.0	375	
6-9	1			217.0			
″	2			218.5	217.8	610	
6-11	1			~~[illegible]~~			Miscount
″	2			118.0			
″	3			118.5	118.2	331	
8-11	1			279.0			
″	2			280.5	279.8	783	
8-9	1			209.0			
″	2			210.0	209.5	587	
5-9	1			215.0			
″	2			216.3	215.7	604	
5-8	1			140.0			
″	2			141.0	140.5	393	
9-11	1			188.0			
″	2			187.5	187.8	526	



PROBLEM A3. AXEMAN AND FLAGMAN PRACTICE.

(a) *Equipment.*—Flag pole, axe, 4 flat stakes, 1 hub, tacks.

(b) *Problem.*—Practice the correct routine duties of axeman and flagman.

(c) *Methods.*—(1) Number three station stakes to indicate representative cases and drive them properly. (2) Drive a hub flush with ground and tack it; number a witness stake and drive it properly. (3) Arrange program of signals with partner, separate 1,000 feet or so and practice same. (4) Signal say five station numbers to each other and afterwards compare notes. Make concise record of the foregoing steps.

PROBLEM A4. RANGE POLE PRACTICE.

(a) *Equipment.*—4 flag poles.

(b) *Problem.*—Given two hubs 1,000 feet or so apart, interpolate a flag pole say 100 feet from one hub, remove the distant pole, prolong the line by successive 100-foot sights and note the error at distant hub. Repeat process for 200-foot and 300-foot sights.

(c) *Methods.*—(1) Set distant flag pole precisely behind hub and hold spike of pole on tack of near hub; lying on ground back of near hub, line in pole 100 feet (paced) distant; remove pole from distant hub, and prolong by 100-foot sights up to distant hub, noting error to nearest 0.01 foot. (2) Repeat in reverse direction, using 200-foot sights. (3) Repeat with 300-foot sights. Avoid all bias. Record data in suitable form, describing steps concisely.

PROBLEM A5. STANDARDIZING CHAIN OR TAPE.

(a) *Equipment.*—Chain or tape assigned in any problem where standard length of chain may be of value.

(b) *Problem.*—Determine the length of the assigned chain or tape by comparison with the official standard under the conditions of actual use.

(c) *Methods.*—In standardizing tape, reproduce the conditions of actual use as regards tension, support, etc., bring one end graduation of chain or tape to coincide with one standard mark, and observe fraction at the other end with a scale. As a general rule, observe one more decimal place than is taken in the actual chaining.

PROBLEM A6. DISTANCES WITH SURVEYORS' CHAIN.

(a) *Equipment.*—Surveyors' chain set of chaining pins, 2 plumb bobs, 2 flag poles, (unless instructed otherwise).

(b) *Problem.*—On an assigned chaining course about one mile long measure distances with the surveyors' chain to the nearest 0.1 link, and repeat the measurements in the opposite direction.

(c) *Methods.*—(1) Standardarize the chain before and after as prescribed in A5. (2) Chain along the assigned course, noting the distances from the starting point to the several intermediate points and to the end station. Observe fractions to the nearest 0.1 link by estimation. (3) Repeat the chaining in the opposite direction, noting the distances from the end point, as before. The difference between the totals

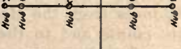
DISTANCES WITH SURVEYORS' CHAIN.

Line	Direction Chained	Observed Length. Ch.	Diff. of Total. Ch.	Ratio 1:d	Coef. C. Lk.
Chain	Before	1.0020			
"	After	1.0022			
A - B	E.	7.327			
A - C	"	30.306			
A - D	"	60.357			
A - E	"	79.838			
E - D	W.	19.479			
E - C	"	49.531			
E - B	"	72.506			
E - A	"	79.833	0.005	1:15970	0.06
		(L)	$E = c\sqrt{L}$ or		$c = \frac{E}{\sqrt{L}}$ (See Diagram)

(Note. The above data will be used in a subsequent problem in discussing the precision of chaining.)

Head Chainman. J. Doe.
Rear Chainman. R. Roe.

Sept 18, 98. (2 hours) Clear and cool.
Used Gunter Chain No 210, Locker No. 38.
Compared chain with official standard both before and after day's chaining.
Chained along Chaining Course "A", beginning at hub with tack, marked A on guard stake, located at S. edge of W. brick walk on Green St. at E. curb line of Mathews Ave, Urbana, Ill.; thence Ely along said S. line of N. brick walk, observing distances to nearest 0.1 lk. to tacked hubs marked B, C, D and E, the total distance from the starting point A being noted.
Chained same course in the reverse direction, carrying total distances from hub E.
Fractions of a link were estimated. Pocket rule was used in standardizing chain.



in the two directions should not exceed 1:5,000. Retain the same party organization throughout the problem. Record the data as in the prescribed form.

PROBLEM A7. DISTANCES WITH THE ENGINEERS' CHAIN.

(a) *Equipment.*—Engineers' chain, set of chaining pins, 2 plumb bobs, 2 flag poles (unless instructed otherwise.)

(b) *Problem.*—On an assigned chaining course about one mile long measure distances with the engineers' chain to the nearest 0.1 foot, and repeat the measurements in the opposite direction.

(c) *Methods.*—(1) Standardize the chain before and after, as prescribed in A5. (2) Chain along the assigned course, noting the distances from the starting point to the several intermediate points and to the end station. Observe fractions to the nearest 0.1 foot by estimation. (3) Repeat the chaining in the opposite direction, noting the distances from the end point, as before. The difference between the totals in the two directions should not exceed 1:5,000. Retain the same party organization throughout the problem. Record the data as in the form opposite.

PROBLEM A8. DISTANCES WITH 100-FOOT STEEL TAPE.

(a) *Equipment.*—100-foot steel band tape with end foot graduated to tenths, set of chaining pins, 2 plumb bobs, 2 flag poles, (unless instructed otherwise).

(b) *Problem.*—On an assigned chaining course about one mile long measure distances with the 100-foot steel band tape to the nearest 0.01 foot, and repeat the measurements in the opposite direction.

(c) *Methods.*—(1) Standardize before and after, as prescribed in A5. (2) Chain along the assigned course, noting the distances from the starting point to the several intermediate points and to the end station. In observing the fractions, first determine the foot units, then estimate the nearest 0.1 foot, then shift the tape along one foot and read the exact fraction on the end of the tape, estimating the

Line	Direction Chained	Observed Length Ft.	Diff. of Total Ft.	Ratio 1:d	Coef. C. Ft.
Chain	Before	100.10			
"	After	100.12			
A-B	E.	484.0			
A-C	"	2002.2			
A-D	"	3987.5			
A-E	"	5274.6			
E-D	W.	1288.9			
E-C	"	3272.4			
E-B	"	4790.2			
E-A	"	5274.3	0.3	1:17580	0.04
		$L = 52.743$	$E = C\sqrt{L}$ or $C = \frac{E}{\sqrt{L}}$ (See Diagram)		

(Note The above data will be used in a subsequent problem in discussing the precision of chaining.)

Head Chainman R. Roe,
Rear Chainman J. Doe.

ENGINEERS' CHAIN.

Sept. 19, '99. (2 hours) Cloudy and cool.
Used 100-Ft. Chain, No 63, Locker No. 35.
Compared chain with official standard both before and after day's chaining.
Chained along Chaining Course "A", beginning at hub with tack, marked A on guard stake, located at S. edge of N. brick walk on Green St. at E. curb line of Mathews Ave, Urbana, Ill; thence E'ly along said S. line of N. brick walk, observing distances to nearest 0.1 Ft. to tacked hubs marked B, C, D and E, the total distance from the starting point A being noted.
Chained same course in the reverse direction, carrying total distances from hub E.
Fractions of a foot were estimated. A pocket rule was used to compare chain with standard.

W ← → E

Line	Direction Chained.	Observed Length. Ft.	Diff. of Total. Ft.	Ratio, 1:d	Coef. C. Ft.
Tape	Before	100.011			
"	After	100.008			
A-B	E.	404.58			
A-C	"	2006.79			
A-D	"	3991.65			
A-E	"	5279.49			
E-D	W.	1287.85			
E-C	"	3275.72			
E-B	"	4794.06			
E-A	"	5279.57	0.08	1:65660	0.012
		$L = 52.7957$	$E = C\sqrt{L}$ or $C = \frac{E}{\sqrt{L}}$ (See Diagram.)		

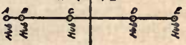
(Note. The above data will be used in a subsequent problem in discussing the precision of chaining.)

Head Chainman J. Doe.
Rear Chainman R. Roe.

100-FT. STEEL TAPE.

Sept. 20. '99. (2 hours.) Clear, moderate.
100-Ft. Roe Steel Tape No. 312, Locker No. 36
Compared tape with official standard both before and after day's chaining measuring fraction with engineers' scale.
Chained Course "A" (previously chained with the Gunter and 100-Ft chains, described on pp. of field note book,) observing continuous distances to hubs B, C, D and E to nearest 0.01 Ft.
Chained course in reverse direction.
Fractions of a foot were estimated to the nearest 0.01 ft. on the end foot of tape which was graduated to tenths of a foot.

W ← → E



nearest 0.01 foot. (3) Repeat the measurement in the opposite direction, noting the distances from the end point, as before. The difference between the totals in the two directions should not exceed 1:10,000. Retain the same party organization. Record data as in form.

PROBLEM A9. HORIZONTAL DISTANCE ON SLOPE WITH STEEL TAPE.

(a) *Equipment.*—100-foot steel tape with etched graduations to 0.01 foot, set of chaining pins, 2 plumb bobs, 3 flag poles, axe, supply of pegs, engineers' level and rod, (unless otherwise instructed).

(b) *Problem.*—Determine the horizontal distance between two assigned points on a steep slope, (1) by direct horizontal measurement, and (2) by measurement on the slope and reduction to the horizontal.

(c) *Methods.*—(1) Standardize the tape for each method, as prescribed in A5, both before and after the day's chaining. (2) In chaining down hill, *rear chairman* lines in flag pole in hand of head chainman, then holds tape end to tack on hub; *flagman* stands 50 feet or more from line opposite middle of tape and directs head chainman in leveling front end, then supports middle point of tape under direction of head chainman; *head chairman*, with spring balance attached to tape and using pole as help to steady pull, brings tension to 12 pounds; *recorder* plumbs down front end, and sets pin slanting sidewise. After checking the pin, proceed with the next 100 feet. In chaining *up hill*, follow same general method, using plumb bob at rear end. In leveling the tape the tendency will be to get the down hill end too low. Chain the line in duplicate, retaining the same organization. (3) Chain the line again in duplicate, tape lying on the ground, pull 12 pounds, pins set plumb, fraction direct to nearest 0.01 foot. Set temporary pegs flush with ground every 100 feet and also at intermediate sudden changes of slope, for levels. Determine differences of elevation between successive pegs, unless the leveling data are supplied to the party. Record data and make reductions and comparisons as in form.

HORIZONTAL DISTANCE △14–△17 (STEEP SLOPE) WITH STEEL TAPE.

Hd. Chain. J. Doe. Flagman. G. W. Sere.
Rr. Chain. R. Roe. Recorder. B. F. Keen.

A. By Direct Horizontal Measurement.

No.	Observed Length. Ft.	Mean Length. Ft	Cor. for Standard Ft.	Reduced Length. Ft.	Diff. (E) Coef. (C) Ratio (1:d)
Tape 1	99.995				
" 2	99.997	99.996			
1	761.45				E = 0.04 C = 0.0015
2	761.49	761.47 / 400000 / −.030	−0.03	761.44	1:23380 (See Diagram)

B. By Measurement on the Slope and Reduction to the Horizontal.

No.	Observed Length. Ft.	Mean Length. Ft.	Cor. for Standard Ft.	Reduced Length. Ft.	Diff. (E) Coef. (C) Ratio (1:d)
Tape 1	100.007				
" 2	100.008	100.008			
1	761.81				E = 0.02 C = 0.007
2	761.79	761.80	+0.06	761.86	1:38590 (See Diagram)
Correction for inclination				−0.47	
Reduced horizontal measm't.				761.39	
Result by first method				761.44	
Difference between results				0.05	= E C = 0.010
Mean of two results				761.42	1:15230

Sept. 21, '99 (3 hours) Cloudy; moderate.

Used 100-Ft. K. & E. etched tape, No. 416, Locker 26, with spring balance.

1st Method Standardized tape (before and after), supported at ends and middle, plumbing ends down, pull 12 lbs.

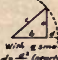
Chained line in duplicate, leveling tape by estimation, pull 12 lbs, plumbing down high end, marking points by chaining pins leaned sidewise.

2nd Method Compared tape with standard (before and after), supported full length on ground, pull 12 lbs.

Chained line in duplicate, tape supported on ground, pull 12 lbs., ends marked by chaining pins. Drove temporary pegs every 100 ft. for levels.

Ran levels over line with following results.

c	a	a^2	d
Ft.	Ft.		Ft.
100	+3.5	12.25	0.06
100	+3.7	13.69	0.07
100	+4.1	16.81	0.08
100	+2.8	7.84	0.04
100	+3.3	10.89	0.05
100	+4.5	20.25	0.10
100	+3.1	9.61	0.05
62	+1.6	2.56	0.02
762			0.47 = Σd



With a small, $d = \frac{a^2}{2c}$ (nearly)

ANGLES OF TRIANGLE 5–6–8 WITH TAPE.

Surveyors { J. Doe R. Roe.

Station	Trig. Function. Name.	Value.	Computed Angle. Half.	Whole.	Mean.
6	Sin ½(6)	0.4050	23°53.5′	47°47′	
"	Tan (6)	1.1023		47°47′	47°47′
8	Sin ½(8)	0.3696	21°41.3′	43°23′	
"	Tan (8)	0.9450		43°23′	43°23′
5	Sin ½(5)	0.6995	~~44°23′~~	~~88°46′~~ (.05)	
"	Tan ½(5)	0.9796	~~44°24.5′~~	~~88°49′~~	
"	Sin ½(5)	0.6997	44°24′	88°48′	
	Tan ½(5)	0.9799	44°25′	88°50′	88°49′
					179°59′
		Error of Closure =			1′
		Permissible Error =			3′

Check with 50-Ft. Metallic Tape.

Angle	Sin ½A	½A	A
6	0.4051	23°54′	47°48′
8	0.3696	21°41.5′	43 23
5	0.6999	44 25	88 50
			180°01′

Assigning equal weight to the three results by steel tape, the most probable values are:

Angle 6 = 47°47.3′
" 8 = 43°23.3′
" 5 = 88°49.4′
180°00′

Sept 22, '99. (2 hours) Clear and warm.

Used Roe 100-Ft. Steel Tape, No. 362, and Lufkin 50-Ft. Metallic Tape, No. 411, Lkr. 36.

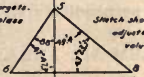
Though not needed in problem, noted the lengths of tapes by standard, 100.01 and 50.01 ft., respectively.

Measured each angle by chord method and checked by tangent method, using radius of 100 ft. with steel tape. In measuring ∠6, (nearly 90°) the tangent method was applied to the bisected angle. Each angle was verified before proceeding to the next, a difference of 2′ being allowed in each.

After an angle was thus verified, a rapid but careful measurement was made with metallic tape, by chord method only, using 50-ft. radius.

Used Flagpoles for distant and pins for close targets.

Used 5-place table.



Sketch shows adjusted values.

Rough test of ∠6, radius = 57.3 ft. Arc = 47.80 ft. ∠6 = 47°48′

PROBLEM A10. ANGLES OF A TRIANGLE WITH TAPE.

(a) *Equipment.*—100-foot steel tape, 50-foot metallic tape, set of chaining pins, 2 plumb bobs, 2 flag poles, five-place tables of trigonometric functions (each member of party to have tables).

(b) *Problem.*—Measure the angles of an assigned triangle with the steel tape and also with the metallic tape, the error of closure not to exceed 3 minutes.

(c) *Methods.*—(1) Measure each angle with the steel tape by both the chord and tangent methods, 100-foot radius, the difference in the two results not to exceed 2 minutes. If the angle is near 90°, the tangent method may be applied to the bisected angle. (2) After securing satisfactory check on an angle with the steel tape, make a rapid but careful measurement with the metallic tape, radius 50 feet. The results may be taken to the nearest half minute. (3) Measure at least one angle, preferably on smooth ground, by laying out an arc with radius of 57.3 feet, setting pins every few feet, and measuring the actual arc. Give close attention to alinement throughout. Record data and make reductions as in form on preceding page.

PROBLEM A11. SURVEY OF FIELD WITH STEEL TAPE.

(a) *Equipment.*—100-foot steel tape, set of chaining pins, 2 plumb bobs, 4 flag poles, five-place table of functions.

(b) *Problem.*—Make survey of an assigned field with tape, collecting all data required for plotting the field and calculating its area by the "perpendicular," "three-side," and "angle" methods.

(c) *Methods.*—Standardize the tape once. (2) Examine the field carefully and plan the survey. (3) Measure the required angles with tape. (4) Locate the perpendiculars. (5) Chain all necessary lines, and also take distances to feet of perpendiculars. Follow form.

PROBLEM A12. AREA OF FIELD BY PERPENDICULAR METHOD.

(a) *Equipment.*—Five-place table of logarithms.

(b) *Problem.*—Calculate the area of the assigned field by

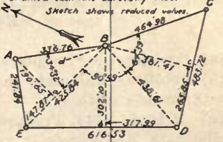
SURVEY OF FIELD A-B-C-D-E WITH TAPE. (DATA FOR AREA AND PLAT.)

Angle	Sin ½A	½A	A	Proof.
ABE	.2968	17°16'	34°32'	
EBD	.7131	45°29.5'	90°59'	190°10'
DBC	.5347	32°19.5'	64°39'	180°00'
ABd	.0888	5°06'	10°12'	10°10'

Line	Observed Length Ft.	Cor for Standard Ft	Reduced Length Ft.
9-25 Tape	99.992		
9-26 Tape	99.960		
AB	336.83	-0.07	336.76
BC	465.07	-0.09	464.98
CD	483.82	-0.10	483.72
DE	616.65	-0.12	616.53
EA	241.89	-0.05	241.84
BE	425.93	-0.09	425.84
BD	430.70	-0.09	430.61
Aa	190.69	-0.04	190.65
Ea	147.90	-0.03	147.87
Bb	302.16	-0.06	302.10
Db	318.05	-0.06	317.99
Bc	381.49	-0.08	381.41
Dc	265.90	-0.05	265.85

Head Chainman. J Doe.
Rear Chainman. A Roe

Sept 25,'99 (3 hours) Cloudy and cool.
Used Roe 100-Ft Steel Tape, No 361, Locker No 35.
Standardized tape before only.
Let fall perpendiculars Aa, Bb and Bc by first estimating the positions of a, b and c, then erecting precise perpendiculars and shifting as required. Set pegs at points a, b and c.
Measured angles ABE, EBD and DBC with tape by chord method, 100-ft radius, and checked by measuring angle between AB and Bd (line CB prolonged)
Sept 26,'99 (2 hours) Drizzling and cold.
Chained each line carefully once.
Sketch shows reduced values.

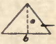


COMPUTATION OF AREA OF FIELD A-B-C-D-E, PERPENDICULAR METHOD.

Sept. 27, '99. Computer J Doe.
Data from pp Transcript checked

Triangle	Line.	Base, b Ft	Altitude, a Ft.	Multiplication	Logarithms	Double Areas Sq Ft.	
ABE	BE	425.84		425.84 / 560.91	2.62926		
	Aa		190.65	42584 / 38326 / 256 / 21	2.28024 / 4.90949		
				81187	(81190)	81190	(Result to nearest 10 Sq Ft.)
BDE	De	616.53		616.53 / 01 203	2.78995		
	Bb		302.10	184959 / 1233 / 62	2.48015 / 5.27010		
				186254	(186250)	186250	
BCD	CD	483.72		483.72 / 14183	2.68459		
	Bc		381.41	145116 / 38698 / 484 / 193 / 3	2.58139 / 5.26598		
				184496	(184500)	184500	
						2)451940	(Result to nearest 0.001 Ac.)
						225970 =	5.188 Ac

Area = ½ab



Note To reduce sq ft. to acres, divide by 43560. Special methods are given below.
1 Acre = 43560 Square Feet = 10 Square Chains

$$1\ Sq\ Ft. = \frac{1}{6\times6\times11\times11\times10}\ Ac. = \frac{1}{43560}\ Ac.$$

$$= 0.0000229596\ Ac.$$

Use long division or one of the methods shown in the application opposite.

Short Division	Contracted Div'n.	Contr'd Mult.
6)225970 6)37661.667 11)6276.944 11)570.631 10)51.876 5.1876	225970)43560 217800 5.1876 8170 4356 3814 3485 329 305 24 26	225970 69592200000.0 45194 4519 2034 113 16 5.1876

the perpendicular method, using the data collected in Problem A11.

(c) *Methods.*—(1) Prepare form for calculation; transcribe data, and carefully verify transcript. (2) Calculate double areas of the several triangles by contracted multiplication, perpendicular method, preserving a consistent degree of precision. (3) Make the same calculations with logarithms, as a check. (4) Combine the verified results, as shown in form.

PROBLEM A13. AREA OF FIELD BY THREE-SIDE METHOD.

(a) *Equipment.*—Five-place table of logarithms.

(b) *Problem.*—Calculate the area of the assigned field by the three-side method.

(c) *Methods.*—(1) Prepare form for calculation; transcribe data, and carefully verify transcript. (2) Calculate the areas of the several triangles by logarithms, three-side method preserving proper units in the results. (3) Carefully review the calculations, and combine the verified results, as in the form opposite.

PROBLEM A14. AREA OF FIELD BY ANGLE METHOD.

(a) *Equipment.*—Five-place table of logarithms.

(b) *Problem.*—Calculate the area of the assigned field by the "two sides and included angle" method, using the data collected in A11.

(c) *Methods.*—(1) Prepare form, transcribe data, and verify copy. (2) Calculate the double areas of the several triangles by contracted multiplication, angle method, preserving consistent accuracy in results. (3) Make same calculations by logarithms, as a check. (4) Combine the checked results. Follow the form opposite.

PROBLEM A15. AREA OF FIELD FROM PLAT.

(a) *Equipment.*—Drafting instruments, paper, etc., planimeter, (as assigned).

(b) *Problem.*—Determine the area of the assigned field directly from the plat.

Sept 28, '99 Computer J. Doe
Data from pp. Transcript checked.

COMPUTATION OF AREA OF FIELD A-B-C-D-E, 3-SIDE METHOD.

Triangle.	Sides Line	Length. Ft	s = ½(a+b+c) Ft	(s−a) Ft.	(s−b) Ft.	(s−c) Ft	Area of Triangle $\sqrt{s(s-a)(s-b)(s-c)}$ Logarithms.	Areas Sq Ft	
ABE	AB = a	338.76	502.22				2.70089		
	BE = b	425.84		16546			2.21869		
	EA = c	241.84			76.38		1.88298		
	2)	100444				260.38	2.41561		
	s =	502 22					2)9.21817		
							4.60908	40650	(To nearest 10 Sq Ft.)
BDE	BD = a	438.81	740.49				2.86952		
	DE = b	616.53		301.88			2.47983		
	EB = c	425.84			123.96		2.09328		
	2)	1480.98				314.65	2.49783		
		740.49					2)9.94046		
							4.97023	93380	
BCD	BC = a	464.98	693.66				2.84115		
	CD = b	483.72		228.66			2.35923		
	DB = c	438.61			209.94		2.32210		
	2)	1387.31				255.05	2.40662		
		693.66					2)9.92910		
							4.96455	92160	
							5.35497	226190 =	5.193 Ac.
							−4.63909	(÷43560)	
							0.71588		

Sept 29, '99. Computer J. Doe
Data from pp. Transcript checked.

COMPUTATION OF AREA OF FIELD A-B-C-D-E, ANGLE METHOD.

Triangle	Part	Value Ft. or ° '	Multiplication a Sin C	ab Sin C	Logarithms	Double Areas		
								Area = ½ ab Sin C
ABE	AB = a	338.76	33676 186650	190.91 48524	2.52732			
	BE = b	425.84	16838 2021	76384 3818	2.62925			
	ABE = C	34°32'	202 27 3	955 153 8	9.75350			
			190.91	81296	4.91007			
					(81300)	81300	(Result to nearest 10 Sq Ft.)	
BDE	BE = a	425.84	42584 589920	42578 16834	2.62925			
	BD = b	438.61	38326 3833	170312 12773	2.64208			
	EBD = C	90°59'	83 34 1 2	3406 255 4	9.99994			
			425.78	186730	5.27127			
					(186758)	186750	. . . . .	
BCD	BD = a	438.81	43861 173020	39638 83464	2.64208			
	BC = b	464.98	39475 132	158552 23783	2.66743			
	DBC = C	64°39'	31	1586	9.95683			
			396.38	357 31	5.26554			
				184303	(184310)	184310	. . . . .	
						2)452360		
					5.35446	226180	= 5.192 Ac.	
					4.63909	(÷ 43580)	(Result to nearest 0.001 Ac.)	
					0.71537	(5.192 Ac.)		

(c) *Methods.*—(1) Make an accurate plat of the field from the notes secured in A11, using a prescribed scale. (2) Determine the area of the field by resolving the polygon into an equivalent triangle. (3) Determine the area from the plat by the polar planimeter and by one of the following "home-made" planimeters: "bird shot" planimeter, "jack knife" planimeter, cross-section paper, parallel strip, weighing, etc. (4) Prepare on the plat a tabulated comparison of results secured by the several methods. (5) Finish the plat, as required.

PROBLEM A16. SURVEY OF FIELD WITH CURVED BOUNDARY.

(a) *Equipment.*—100-foot tape, 50-foot metallic tape, set of chaining pins, 2 plumb bobs, 4 flag poles.

(b) *Problem.*—Make survey with tape of an assigned tract having a curved boundary, collecting all data required for plotting the field and calculating its area.

(c) *Methods.*—(1) Standarize the tape once to nearest 0.01 foot. (2) Examine the tract carefully and plan the survey so as to secure a simple layout of base lines designed to give short offsets to the curved boundaries. (3) Locate the perpendiculars, if any, and chain all lines; on the curved sides, take offsets so as to secure a definite location, and as a rule take equal intervals on the same line. Follow the form opposite.

PROBLEM A17. AREA OF FIELD WITH CURVED BOUNDARY.

(a) *Equipment.*—(No instrumental equipment required).

(b) *Problem.*—Calculate the area of the assigned field with curved boundary by "Simpson's one-third rule", using the data collected in Problem A16.

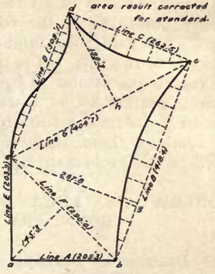
(c) *Methods.*—(1) Prepare form for calculation; transcribe data in convenient form for calculation, and carefully check copy. (2) Calculate the area of the polygon formed by the base lines, preferably by the perpendicular method. (3) Calculate the areas of the curved figures by "Simpson's one-

SURVEY OF FIELD WITH CURVED BOUNDARY LINE.

Offset L. Ft.	Dist. Ft.	Offset R. Ft.	Offset L. Ft.	Dist. Ft.	Offset R. Ft.
0	262.5	= d			
13.5	240				
30.3	200		e =	309.1	0
39.0	160			300	2.1
39.4	120			280	8.5
31.0	80			260	13.2
19.6	40			240	14.7
0	0	= c		220	15.0
	Line C	(c to d)		200	14.8
				180	10.0
0	418.4	= c		160	2.0
9.5	400		0	154.3	0
24.6	360		7.2	140	
32.4	320		15.0	120	
39.3	280		19.7	100	
40.7	240		20.8	80	
40.3	200		20.2	60	
37.4	160		18.4	40	
30.1	120		10.3	20	
21.2	80		0	0	= d
10.8	40			Line D	(d to e)
0	0	= b	Tape =	100.01	
	Line B (Read up)	(b to c)	Oct.3,'99	Clear and warm. (Read up.)	

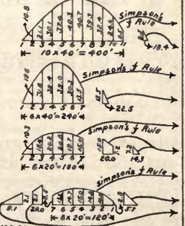
Head Chainman, R. Roe.
Rear Chainman, J. Doe.

Oct. 2,'99 (3 hours) Clear and warm.
Tape No. 361, Locker No. 35 = 100.01'.
Sketch shows observed lengths. Final area result corrected for standard.



COMPUTATION OF AREA OF FIELD WITH CURVED BOUNDARY.

Oct. 8,'99 Computer, J. Doe.
Data from pp. Transcript checked.



Part.	Data for Calculation of Areas.	Indicated Calculations.	+Areas	−Areas
abe	Triangle, Base = 290.'0, Alt. = 145.'3	$\frac{1}{2}(290.0 \times 145.3)$	21068	
bce	" " = 418.'4 " = 267.'8	$\frac{1}{2}(418.4 \times 267.8)$	36024	
cde	" " = 404.'7 " = 199.'3	$\frac{1}{2}(404.7 \times 199.3)$	40328	
Line B		$\frac{40}{3}[(0+9.5)$ $+2(21.1+37.4+40.7+32.4)$ $+4(10.8+30.1+40.3+39.3+24.6)]$		11375
		$\frac{1}{2}(9.5 \times 18.4)$		87
Line C		$\frac{40}{3}[(0+13.5)$ $+2(31.8+39.0)$ $+4(19.6+39.4+30.3)]$		6831
		$\frac{1}{2}(13.5 \times 22.5)$		152
Line D		$\frac{20}{3}[(0+15.0)$ $+2(18.4+20.8)$ $+4(10.3+20.2+19.7)]$		1361
		$\frac{1}{2}[20(15.0+7.2)+(7.2 \times 14.3)]$		273
" "		$\frac{20}{3}[(2.0+8.5)$ $+2(14.8+14.7)$ $+4(10.0+15.0+13.2)]$	1407	
		$\frac{1}{2}(2.0 \times 5.7)$	6	
		$\frac{1}{2}[(2.1 \times 9.1)+20(2.1+8.5)]$	116	
Chain = 100.01		Chain Cor. { 98352 / 2000.0 / +20	119031 / 20679 / 98352 / +20 / 98372 = 2.258 Ac.	20679

True Area = Comptd Area × $(1+0.0001)^2$
$(1+0.0001)^2 = (1+0.0002)$ (nearly)

third rule," which is as follows: "Divide the base line into an *even number of equal parts* and erect ordinates at the sum by one-third of the common distance between ordinates, twice the sum of all the other odd ordinates, and four times the sum of all the even ordinates; multiply the sum by one-third of the common distance between ordi nates." The field notes might have been taken with special reference to the rule, but it is better to take from the notes the largest *even number* of equal segments, assuming the remaining portion to be a trapezoid or triangle. (4) Give signs to the several results by reference to the field sketch, and combine them algebraically to get the net area, as shown in the accompanying form.

PROBLEM A18. AREA OF FIELD WITH CURVED BOUNDARY FROM PLAT.

(a) *Equipment.*—Drafting instruments, paper, etc., planimeter (as assigned).

(b) *Problem.*—Determine the area of the field with curved boundary directly from the plat.

(c) *Methods.*—(1) Make an accurate plat of the field from the notes obtained in A16, using a prescribed scale. (2) Determine its area directly from plat by two methods mentioned in (3) of A15, other than those used in that problem. (3) Prepare on the plat a tabulated comparison of the results by the several methods. (4) Finish the plat, as required.

PROBLEM A19. PASSING AN OBSTACLE WITH TAPE.

(a) *Equipment.*—100-foot steel tape, set of chaining pins, plumb bobs, 4 flag poles.

(b) *Problem.*—Prolong an assigned line through an assumed obstacle by one method and prove by another, finally checking on a precise point previously established.

(c) *Methods.*—Given two hubs, A and B, 200 feet apart, prolong line and establish C 200 feet from B: (1) by constructing a 200-foot square in one direction; and (2) by laying off a 200-foot equilateral triangle on the opposite side, using pins to mark points thus established. (3) Prolong the

PASSING OBSTACLE

Oct. 4, '39 (2 hours) Clear and warm.

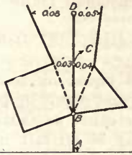
Tape No. 361, Locker No. 35, Length = 100.'01.

Given three hubs, (set on true line by transit), B 200 ft. from A, and D 400 ft. beyond B, all on smooth ground.

Assumed obstacle as shown in sketch, and then (ignoring D) passed obstacle by 200-ft. equilateral triangle to right and by 200-ft. square to left. Resumed line by each method and prolonged to point D. Used pins marked by slips of paper to indicate points.

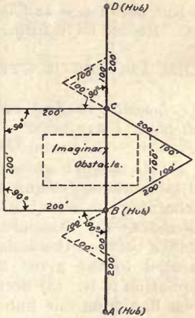
Also interpolated C on BD carefully by eye.

Results are given in diagram below.



Chainmen. { J. Doe / R. Roe

WITH STEEL TAPE.



OBSTRUCTED DISTANCE

Oct. 5, '39 (2 hours) Cloudy and cool.

Tape No. 361, Locker No. 35, Length = 99.'99.

Given two hubs A and B an unknown distance apart, on smooth ground.

Assumed an obstruction to vision and measurement, as shown in sketch.

Selected point C visible from A and B, chained CA and CB, observing nearest 0.1 ft., and bisected CA at D and CB at E. Chained DE. Then calculated AB by doubling ED. 260.7 × 2 = 521.4

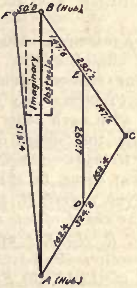
Ran random line from A as close as practicable to obstruction so as to reduce BE to a minimum. Let fall perpendicular BF from B on random line. Measured BF and FA to nearest 0.1 ft. Calculated hypothenuse AB.
$AB = \sqrt{50.8^2 + 519.4^2} = 521.'0.$

Finally, after securing the above results, chained the actual distance AB. The three results are summarized below.

Method.	Obs. Dist.	Std. Cor.	Red. Dist.
By similar triangles	521.'4	-0.'1	521.'3
By right triangle	521.0	-0.1	520.9
By actual meas'mt	521.5	-0.1	521.4
Total range = 1:1040			

Chainmen, { J. Doe / R. Roe

WITH STEEL TAPE.



line by each method to the hub D, 200 feet from C, and record discrepancies in line. (4) Interpolate a point at C on true line between B and D, and note errors of prolongation at C. Record as in form.

PROBLEM A20. OBSTRUCTED DISTANCE WITH TAPE.

(a) *Equipment.*—100-foot steel tape, set of chaining pins, 2 plumb bobs, 4 flag poles.

(b) *Problem.*—Determine the distance between two assigned points through an assumed obstruction to both vision and measurement, using two independent methods, and finally chaining the actual distance.

(c) *Methods.*—(1) Standardize the tape. (2) Determine the distance between the assigned points by constructing a line parallel to the given line, and equal or bearing a known relation to it. (3) Secure a second result by running a random line from one hub past the other so that a perpendicular less than 100 feet long may be let fall, measuring the two sides and calculating the hypothenuse. (4) After securing two results differing by not more than 1:1,000, chain the actual distance. Follow form.

PROBLEM A21. RUNNING IN CURVE WITH TAPE.

(a) *Equipment.*—100-foot steel tape, 50-foot metallic tape, set of chaining pins, 2 plumb bobs, 3 hubs, 6 flat stakes, marking crayon, tacks, five-place table of functions.

(b) *Problem.*—Lay out two lines making an assigned angle with each other, and connect them with a prescribed curve by the "chord offset" method.

(c) *Methods.*—(1) Calculate the *radius, R,* for the given *degree of curve, D.* (2) Calculate the *tangent distance, T,* for the given *radius, R,* and *angle of intersection, I.* (3) Calculate the *chord offset, d,* and *tangent offset, t,* for the known *radius, R, chord, c* and *degree, D.* (4) At the given point intersection (P. I.), A, lay off the given angle, *I,* by the chord method. (5) From the P. I. lay off *T* along the two tangent lines and locate point tangent (P. T.) and point curve (P. C.), setting hubs at P. C. and P. T., with guard stake at each hub. (6) Run in the curve, by chord offsets,

LOCATION OF CURVE

Oct. 6, '39. (3 hours.) Clear and cool.

100-Ft Steel Tape No. 361, Locher 35 = 100.00

Given hub at A and a distant hub B, to lay off a line AC making an angle I of 80° with BA prolonged, and connect the two lines with a 20° degree curve, that is, a curve having a central angle of 20° subtended by a 100-ft. chord, c.

The radius was calculated thus: Since the chord of an arc is twice the sine of half the arc, chord = 2 rad. × sin ½ D

$$rad = \frac{\frac{1}{2} chord}{\sin \frac{1}{2} D} = \frac{50}{0.17365} = 287.9$$

Calculated tangent distance thus: In right triangle (O-P.C.-P.I.)

Tan. Dist = Rad. × Tan ½ I

= 287.9 × 0.83910 = 241.6

Calculated chord offset d, and tangent offset t, thus: By similar triangles, R:c::c:d

$d = \frac{c^2}{R} = \frac{100^2}{287.9} = 34.97$, $t = \frac{1}{2} d = 17.48$

(An approx. formula is $d = 1\frac{3}{4}D = 35$, $t = \frac{7}{8}D = 17.5$)

From A (Point intersection) laid off Tan. Dist. (T), locating Point Curve (P.C.) and Point Tangent (P.T.). Began at P.C. and ran in curve, as shown in sketch. Error of closure at P.T. was 0.2 in line and 0.1 in distance.

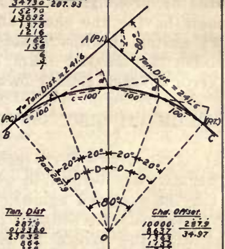
Head Chain. J. Doe. Rear Chain. R. Roe

Axeman. B. F. Keen. Flagman. G. W. Sure.

WITH STEEL TAPE.

Rad

50.000 ÷ 0.17365 = 287.93



Tan. Dist

287.9 × 0.83910 = 241.58

Chd. Offset.

10000 ÷ 287.9 = 34.97

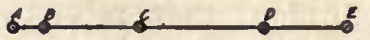
DISCUSSION OF ERRORS

Line	Direction Chained	Observed Length Ft.	Difference E. Ft.	Chaining Ratio 1 : d.	Coef. of Precision (C.) Ft.
A-B	E.	484.58	E +, W −		
B-A	W.	484.61	−0.03	0.000062 1:16150	0.014
A-C	E.	2003.79			
C-A	W.	2003.85	−0.06	0.000029 1:33400	0.013
A-D	E.	3991.69			
D-A	W.	3991.74	−0.05	0.000012 1:79830	0.008
A-E	E.	5279.48			
E-A	W.	5279.57	−0.09	0.000017 1:58660	0.012
B-C	E.	1519.21			
C-B	W.	1519.24	−0.03	0.000019 1:50640	0.008
B-D	E.	3507.11			
D-B	W.	3507.13	−0.02	0.000006 1:175350	0.003
B-E	E.	4794.90			
E-B	W.	4794.96	−0.06	0.000012 1:79850	0.009
C-D	E.	1987.90			
D-C	W.	1987.89	+0.01	0.000005 1:198790	0.002
C-E	E.	3275.69			
E-C	W.	3275.72	−0.03	0.000009 1:109190	0.005
D-E	E.	1287.79			
E-D	W.	1287.83	−0.04	0.000030 1:32920	0.011
		(L in	Σ =	0.000202	0.085
		100-ft.	Mean =	.0000202 1:49500	0.008
		units.)	$E = C\sqrt{L}$ or	$C = \frac{E}{\sqrt{L}}$ (See Diagram)	

Oct. 9, '39 Computer J. Doe

Data from pp. Transcript O.K.

WITH STEEL TAPE.



Distances by Subtraction.

B-A 5279.57	E-A 5279.57	E-A 5279.57
E-B 4794.96	E-C 3275.72	E-D 1287.83
B-A 484.61	C-A 2003.85	D-A 3991.74
A-C 2003.79	E-B 4794.96	A-D 3991.69
A-B 484.58	E-C 3275.72	A-B 484.58
B-C 1519.21	C-B 1519.24	B-D 3507.11
E-B 4794.96	A-E 5279.48	A-D 3991.69
E-D 1287.83	A-B 484.58	A-C 2003.79
D-B 3507.13	B-E 4794.90	C-D 1987.90
E-C 3275.72	A-E 5279.48	A-E 5279.48
E-D 1287.83	A-C 2003.79	A-D 3991.69
D-C 1987.89	C-E 3275.69	D-E 1287.79

Designating E + and W − (4th Column) it is seen that the returning results (except CD) are greater. This is explained by standard tape lengths, viz., before = 100.011, after = 100.008, i.e. the tape gradually decreased in length, causing greater observed lengths.

beginning at P. C. and checking at P. T. Calling P. C. Station 0, establish Station 1 by laying off tangent offset, t, and chord, c. Having one station on the curve, the next is located by prolonging the chord and forming an isosceles triangle having the chord offset as a base. Check on the P. T., noting the discrepancy of distance and line. Also establish the tangent again by tangent offset and observe the error of line. Follow form.

PROBLEM A22. DISCUSSION OF ERRORS OF CHAINING.

(a) *Equipment.*—(No instrumental equipment, unless further data are desired, in which case Problems A6, A7 and A8 may be assigned again).

(b) *Problem.*—Investigate the errors of linear measurement with the several kinds of chains and tape, with the view to determine practical working tests or coefficients of precision for actual use.

(c) *Methods.*—Assume that the conditions in Problems A6, A7 and A8 are practically constant in the same problem, and that the actual differences between observed lengths of the several segments when chained in opposite directions, represent the normal errors with the particular chain and chainmen; then tabulate: (1) the measured lengths of all possible segments of the chaining course, either from direct observation or by subtraction; (2) the actual errors or differences between the two results, giving signs; (3) the chaining ratios, $1:d$, and the decimal expressions of the same to six places; (4) the "coefficients of precision" for each case, calculated by formula, or more quickly, taken from the diagram in the chapter on errors of surveying; (5) the mean decimal chaining ratio and its equivalent; and (6) the mean coefficient of precision. Follow the prescribed form.

PROBLEM A23. TESTING (OR ESTABLISHING) AN OFFICIAL STANDARD OF LENGTH.

(a) *Equipment.*—Standard tape (with certified length given), turnbuckle adjustments with bolts, spring balance, standard steel rule graduated to 0.01 inch, 2 thermometers, 2 microscopes, strips of wood, a watch.

(b) *Problem.*—Make a series of ten observations with a standardized steel tape for the purpose of testing (or establishing) an official standard of length, observing the nearest 0.0001 foot.

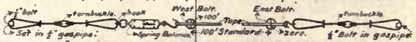
(c) *Methods.*—(If a *new* offical standard is being established, one standard mark may be made permanent, and the precise distance taken to an approximate temporary point on the other bolt, the exact correction being applied after a sufficient number of results have been obtained. If the sun is shining, the tape should be protected by a wooden box or other covering throughout its length. Cloudy days or night time give best results. The observations should be made briskly so as to have slight range of temperature. If isolated standard monuments are used, their foundation should go below frost line, and the monuments should be located so as to suffer as little as possible from heaving. If the standard marks are indoors, the conditions are less difficult to control).

(1) Arrange "bucksaw" or turnbuckle adjustments, each held firmly by a bolt dropped into a piece of gaspipe driven

Oct 10, 99. Cloudy and cool. Party { 1. J. Doe. 2. R. Roe. 3. B. F. Keen 4. G. W. Sure.

TEST OF 100-FT. STANDARD, ———— UNIVERSITY.

Selected cloudy day with slight range of temperature during period of observations. Used Standard Tape, No. 417, marked "U.S.W. & M. 215," certified length = 99.9967 ft., at 62° F. with 12-lb. pull, tape supported, coefficient of expansion = 0.0000061.



Program. Arranged "bucksaw" adjustments, etc., as shown in sketch, tape supported on strip of wood. (a) Doe set zero of tape at east standard mark with reading glass. (b) Doe set balance at 12 lbs. (c) Keen observed fraction at west standard mark, using Starrett steel scale graduated to 0.01 in., estimating to nearest 0.001 in. with reading glass. (d) Sure recorded all data and observed time and temperature, two thermometers placed one each at 33' and 67.' Released pull between observations.

No.	Time	Temperature.				Temp. Cor.	Tape.	West Fraction.		Standard	Prob. Error.	
	P.M.	At 33'	At 67'	Mean.	62°-M'n	Ft. −	Ft.	In. +	Ft. +	Ft.	d(.0001)	d^2
1	2:23	52°.0	53°.0	52°.5	9°.5	0.0058	99.9909	0.116	0.0097	100.0006	1	1
2	:28	52.0	53.0	52.5	9.5	.0058	99.9909	.118	.0098	100.0007	0	0
3	:32	52.0	53.0	52.5	9.5	.0058	99.9909	.116	.0097	100.0006	1	1
4	:35	52.0	53.0	52.5	9.5	.0058	99.9909	.118	.0098	100.0007	0	0
5	:39	52.0	52.5	52.2	9.8	.0060	99.9907	.121	.0101	100.0008	1	1
6	:46	52.0	52.5	52.2	9.8	.0060	99.9907	.120	.0100	100.0007	0	0
7	:53	52.5	52.5	52.5	9.5	.0058	99.9909	.119	.0099	100.0008	1	1
8	:58	52.5	52.0	52.2	9.8	.0060	99.9907	.122	.0102	100.0009	2	4
9	3:04	52.0	52.0	52.0	10.0	.0061	99.9906	.121	.0101	100.0007	0	0
10	:08	52.0	52.0	52.0	10.0	.0061	99.9906	.122	.0102	100.0008	1	1

n.

$E_1 = 0.67\sqrt{\frac{\Sigma d^2}{n-1}} = 0.000067$ $E_m = \frac{E_1}{\sqrt{n}} = 0.000021$

Mean = 100.0007 $\Sigma d^2 = 9$

Length of Standard = 100.0007 ± 0.00002 Ft.

flush with surface of ground, with spring balance and tape lined up, as shown in sketch in accompanying form; place the two thermometers at the one-third points as nearly as possible under the actual conditions of the tape. (2) With four men in party, No. 1 sets end graduation precisely at one standard mark by means of screw adjustments and microscope; No. 2 sets balance at 12 pounds; No. 3 observes fraction at other standard mark by means of steel scale graduated to 0.01 inch, estimating to nearest 0.001 inch (say 0.0001 foot) by microscope; and No. 4 records all data, observes time to nearest minute, and temperature to nearest 0.1 degree. Nos. 1, 2 and 3 should lie flat. Release the tension between observations. Record and reduce as in form.

PROBLEM A24. DETERMINATION OF CONSTANTS OF A STEEL TAPE.

(a) *Equipment.*—Steel tape and other articles named in preceding problem.

(b) *Problem.*—Determine coefficients of expansion and stretch of the assigned tape.

(c) *Methods.*—(To be devised by the student.)

PROBLEM A25. COMPARISON OF DIFFERENT MAKES AND TYPES OF CHAINS AND TAPES.

(a) *Equipment.*—Department equipment and collection of catalogs of representative instrument makers.

(b) *Problem.*—Make a critical comparison of the several types of chains and tapes made by different makers.

(c) *Methods.*—Study the different catalogs and prepare a systematic and concise report.

CHAPTER III.

THE COMPASS.

Description.—The magnetic compass consists of a line of sight attached to a graduated circular box, at the center of which is a magnetic needle supported on a steel pivot. The compass box is attached to a tripod or jacob staff by a ball and socket joint, and is leveled by means of the plate levels. The needle should be strongly magnetized and have an agate cap to receive the point of the hardened steel pivot. The dip of the needle is counter-balanced by a small coil of wire, which can be shifted as desired. The E and W points are reversed.

In Fig. 10 are shown the usual types of magnetic compasses: (a) the vernier compass; (b) the plain compass; (c) the vernier pocket compass with folding sights; (d) the ordinary pocket compass; (e) the prismatic compass.

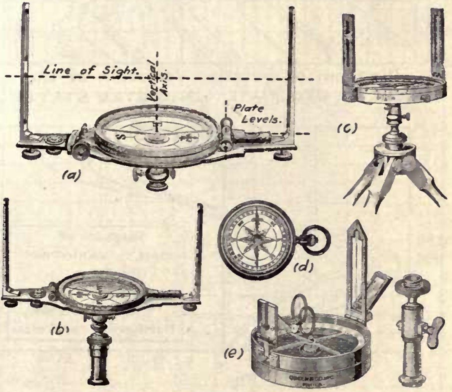


Fig. 10.

Declination of the Needle.—If the needle is allowed to swing freely, its magnetic axis will come to rest in the magnetic meridian. The horizontal angle between the magnetic meridian and the true meridian at any point is called the *magnetic declination* for that point. Imaginary lines joining points on the earth's surface having the same declination are called *isogonic lines*. The isogonic line joining the points of zero declination is called the *agonic line*. Fig. 12 is an isogonic chart of the entire earth's surface. Of the three isogonic lines, one passes through Michigan, Ohio, etc.

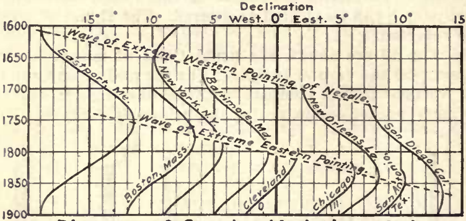


Diagram of Secular Variation of the MAGNETIC DECLINATION IN UNITED STATES.

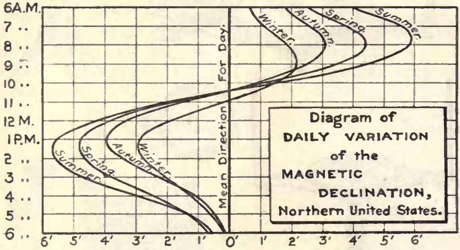


Fig. 11.

Variation of the Declination.—The declination of the needle is not a constant at any place. The change or fluctuation is called the *variation* of the declination. The variations of the magnetic needle are of several kinds: secular, daily, annual, lunar, and irregular variations due to magnetic storms. The most important of these is the *secular* variation which is illustrated in the upper diagram

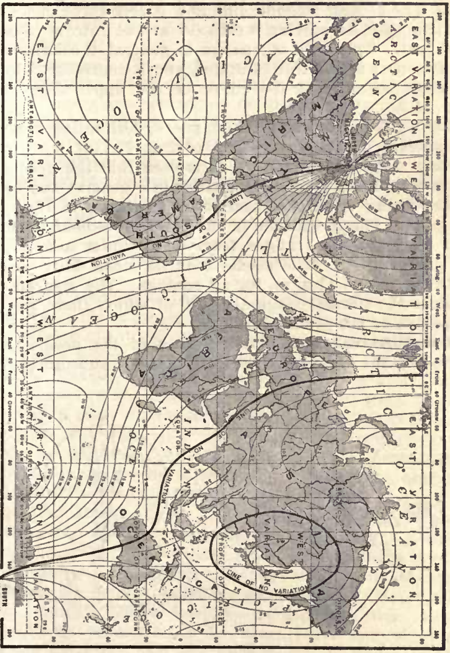


Fig. 12. CHART OF LINES OF EQUAL MAGNETIC DECLINATION.

of Fig. 11 for a series of representative points in the United States. This diagram shows that the extreme range or swing of the needle is roughly 6° or 7°, and that the period of time between extreme positions is about a century and a half. Also that the wave of magnetic influence progresses across the continent alike in successive cycles. At present (1900) the needle is at its extreme western position at Eastport, Me., and at its extreme eastern pointing at San Diego, Cal. The 3° East isogonic line now passes through western Indiana, and is moving westward at the rate of about 4′ per year. This rate of change is general throughout the central part of the United States, and is represented by the straight sections of the curve in the upper diagram of Fig. 11.

The *daily* variation of the magnetic declination is shown graphically in the lower part of Fig. 11, the scale being greatly magnified laterally. It is seen that the needle undergoes each day a vibration similar in a general way to the grand swing of three centuries or so shown in the upper diagram. The magnitude of the daily movement in northern United States ranges from 5′ in winter to nearly 12′ in summer time. The needle is in its mean daily position between 10 and 11 a. m. for all seasons. The diagram represents the normal magnetic day, of which there are perhaps five or six per month.

Local Attraction.—The pointing of the needle is affected by the close proximity of magnetic substances, such

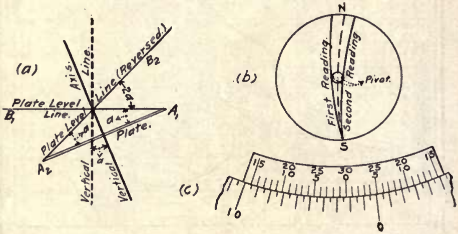


Fig. 13.

as iron ore, wire fences, railroad rails, etc. However, local attraction does not prevent correct work, provided back and fore sights are taken without change of magnetic conditions. It is therefore especially important to avoid disturbances of the needle by the chain, axe, passing vehicles, electric wires, etc., or by articles on the person of the observer, such as keys, knife, spectacle frame, wire in the hat rim, reading glass case, etc. Also the glass cover may become electrified by friction and attract the needle, in which case it may be discharged with the moistened finger, or by breathing on it.

The Vernier.—The vernier is an auxiliary scale used to read fractional parts of the divisions of the main scale or limb. Verniers are retrograde or direct, according as the divisions on the vernier are larger or smaller than those on the limb. The vernier used on compasses for the setting off of the declination is direct, and is usually of the type shown in (c) of Fig. 13. In reading a vernier of any kind, blunders may be avoided by first estimating the fraction by eye before noting the matched lines on the two scales.

USE OF THE COMPASS.

Use.—The compass is used: (1) to determine the bearings of lines; (2) to measure the angle formed by two lines; (3) to retrace old lines. The bearing of a line is the horizontal angle between the line and a meridian through one end of it. Bearings are measured from the north or south point 90° each way. The angle between two lines is the difference in their directions as indicated by the bearings. Having the true bearings of one side of a polygon, the true bearings of the others may be obtained by algebraic addition of the angles; or by using the declination vernier so as to read the true bearing direct on the fore sights.

Practical Hints.—Point the north end of the compass box along the line and read the north end of the needle. Protect the pivot from needless wear by turning the needle in about the proper direction before releasing it. Always lift the needle before disturbing the compass. Habitually obtain duplicate needle readings on each sighting. Read the needle by estimation to the nearest five minutes, that is, to the one-sixth part of one-half degree, which is the usual subdivision of the compass box. Care should be taken to avoid parallax in reading the needle.

ADJUSTMENTS AND TESTS.

Elementary Lines.—The *elementary lines* of the compass, shown in (a) of Fig. 10, are: (1) the line of sight; (2) the vertical axis; (3) the plate level lines.

The maker should see: (1) that the needle is strongly magnetized; (2) that the magnetic axis corresponds with the line joining the two ends; (3) that the metal in the compass box is non-magnetic; (4) that the line of sights passes through the center of graduation; (5) that the plates are perpendicular to the vertical axis; (6) that the zero of the vernier coincides with the line of sights.

The needle may be magnetized with a bar magnet or by putting it into the magnetic field of a dynamo. The metal of the compass box may be tested by reading the needle, then moving the vernier and noting if the needle has moved the same amount, this process being repeated at intervals around the full circle.

The Principle of Reversion.—In adjusting surveying instruments, the presence, direction and amount of the error are made evident by the *method of reversions* which doubles the apparent error. If there is no difference after reversion, there is no error.

Plate Levels.—*To make the plane of the plate level lines perpendicular to the vertical axis.*—Level up the instrument by means of the plate levels and reverse the compass box in azimuth, that is, turn it through a horizontal angle of 180°. Correct one-half the error, if any, by means of the adjusting screws at the end of the level tube, and bring the bubble to the center by the ball and socket joint. The reasons for this process are shown in (a) of Fig. 13.

Sights.—*To make the plane of sights normal to the plane of the plate level lines.*—With one sight removed and the instrument leveled, range in with the remaining sight two points as far apart vertically as possible, say on the side of a building. Reverse in azimuth and bring the bottom of the sight in range with the lower point; if the upper point is then in range, the sight is in adjustment. If not, correct one-half the error by putting paper under one side, or by filing off the other side. Repeat process for the other sight.

The Pivot.—*To adjust the pivot to the center of the graduated circle.*—Set the south end of the needle to read zero, and read the north end of the needle; reverse the compass box in azimuth, repeat the observations, and correct one-half the difference between the two readings of the north end of the needle by bending the pivot, using the special wrench for the purpose. Turn the compass box 90° and repeat. See (b), Fig. 13.

The Needle.—*To straighten the needle.*—Having adjusted the pivot, set the north end of the needle to read zero and bend the needle so that the south end reads zero also. Turn the compass box and test for other graduations.

PROBLEMS WITH THE COMPASS.

PROBLEM B1. DECLINATION OF THE MAGNETIC NEEDLE.

(a) *Equipment.*—Surveyors' compass, flag pole, reading glass.

DECLINATION OF NEEDLE WITH SURVEYORS' COMPASS.

No.	Needle Reading	Mean	Time P.M.	Mean P.M.
1	N3°30'E		2:05	
2	N3°35'E		2:11	
3	N3°30'E		2:15	
4	N3°30'E		2:22	
5	N3°35'E		2:27	
6	N3°35'E		2:31	
7	N3°35'E		2:35	
8	N3°30'E		2:42	
9	N3°35'E		2:48	
10	N3°35'E	N3°33'E	2:54	2:30

(Note. Assuming that the magnetic conditions are normal for the day, the correction for daily variation by Diagram of Daily Variation is 3 minutes West, which added to the mean gives N 3°36'E as the most probable value of the declination for this particular instrument.)

Oct. 12, '00. (2 hours) Clear and cool.

Used Gurley Compass No. 26 (Needle recently remagnetized), and Good Watch.

Set Compass on true meridian with declination vernier set to read zero.

Sighted on flag pole set on meridian at a distance of 200 ft., and read needle by estimation to 5 minutes (one-sixth part of one-half degree), carefully avoiding parallax and magnetic disturbances. Observed time to nearest minute.

Disturbed needle by lifting it from pivot and verified sighting; then when oscillations had ceased to read the needle.

Continued the process until ten consecutive readings, having a maximum range of not more than ten minutes, were obtained.

(b) *Problem.*—At a point on the true meridian determine the mean magnetic declination with the surveyors' compass.

(c) *Methods.*—(1) Set the compass over one point and a flag pole at another on the true meridian. (2) Lower the needle and sight at the flag pole carefully with the north end of the compass box to the front. (3) When the vibrations of the needle have ceased, move the vernier by means of the tangent screw so that the north end of the needle reads zero, and check the sighting of the compass. (4) Read the declination on the vernier to the nearest minute. (5) Lift the needle, verify the zero needle reading and the sighting, read the vernier and record; repeat the process until ten satisfactory consecutive values of the declination are obtained. Observe the time of each reading to the nearest minute. (6) Correct the mean of the ten values for daily variation by reference to the diagram, Fig. 11, using the mean time. Record and reduce the data as in form. (Note that the values in the form were obtained by estimating the nearest five minutes. Which is better? Try both if time allows.)

PROBLEM B2. ANGLES OF TRIANGLE WITH COMPASS.

(a) *Equipment.*—Surveyors' compass, two flag poles, reading glass.

(b) *Problem.*—Measure the angles of a given triangle with the surveyors' compass.

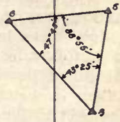
(c) *Methods.*—(1) Set the compass over one of the vertices of the triangle and a flag pole behind each of the other two. (2) Lower the needle and sight at one of the flag poles carefully, with the north end of the box to the front. (3) When the vibrations have ceased, read the north end of the needle to the nearest five minutes by estimation. (4) Lift the needle, verify the sighting and also the reading. (5) Turn the compass box to the other point and determine the bearing, as before. The required angle is the difference between the two bearings. (6) Measure the other two angles in like manner. The error of closure must not exceed 5 minutes. Follow the prescribed form.

ANGLES OF TRIANGLE 5-6-8 WITH SURVEYORS' COMPASS.

Station	Line	Observed Bearing	Needle Angle
5	5-6	S 85°15' W	
"	5-8	S 5°40' W	88°55'
8	8-5	N 5°40' E	
"	8-6	N 45°05' W	45°25'
6	6-8	S 49°00' E	
"	6-5	N 85°15' E	47°45'
			180°05'

Oct. 13, '99. (2 hours) Clear, moderate.
Used Gurley Compass, Locker No. 86.
Each bearing was observed in duplicate, the needle being disturbed and the sighting verified between readings.

(Discrepancy not to exceed 5')



Observers { J. Doe. / R. Roe.

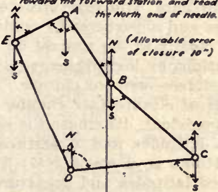
TRAVERSE OF FIELD A-B-C-D-E WITH COMPASS AND CHAIN.

Station	Line	Observed Bearing	Interior Angle	Adjusted Bearing	Distance Ft.
A	AE	S 60°30' W	83°15'		
	AB	S 32°45' E		S 32°45' E	336.5
B	BA	N 33°05' W	180°20'		
	BC	S 43°25' E		S 43°05' E	464.6
C	CB	N 43°20' W	55°05'		
	CD	S 81°35' W		S 81°50' W	483.3
D	DC	N 81°55' E	103°55		
	DE	N 22°20' W		N 22°05' W	816.0
E	ED	S 22°20' E	97°30'		
	EA	N 60°10' E		N 60°25' E	241.6
			540°05'		

See calculation of latitudes and departures on pp.
See calculation of area on pp.

Oct. 16, '98, (3 hours), Clear and windy.
Used Gurley Compass, Locker No. 24.
Made needle read zero when pointing true North by setting off declination with vernier on declination arc of N. 3°36' E.
Read bearings with North end of Compass toward the forward station and read the North end of needle.

(Allowable error of closure 10')



Chained each line once with Engineers' Chain. Length of chain = 100.08 ft.

PROBLEM B3. TRAVERSE OF FIELD WITH COMPASS.

(a) *Equipment.*—Surveyors' compass, 2 flag poles, engineers' chain, set of chaining pins.

(b) *Problem.*—Determine the bearings of the sides of an assigned field with the surveyors' compass and measure the lengths of the sides with an engineers' chain.

(c) *Methods.*—(1) Set the compass over one of the corners of the field which is free from local attraction, and set off the declination with the vernier. (2) Take back sight on the last point to the left and fore sight to the next point to the right, following the methods used in Problem B2. (3) Repeat this process for the remaining corners of the polygon taken in succession to the right. (4) Chain the sides of the field to the nearest 0.1 foot by estimation. (5) Compare the chain with standard. (6) From the observed bearings compute the interior angles of the field, and the true bearings of the sides. The angular error of closure must not exceed 10 minutes for a five-sided field . Record and reduce data as in prescribed form.

PROBLEM B4. AREA OF FIELD WITH COMPASS.

(a) *Equipment.*—Five-place table of logarithms.

(b) *Problem.*—Compute the area of the assigned field by means of latitudes and departures.

(c) *Methods.*—(1) Prepare forms for calculation; transcribe data, and carefully verify copy. (2) Compute latitudes and departures by contracted multiplication, preserving results to the nearest 0.1 foot. (3) Make the same calculations by logarithms, as a check. (4) Determine the actual linear error of closure. (5) Determine the permissible error of closure (see chapter on errors of surveying). (6) If consistent, distribute the errors in proportion to the several latitudes and departures, respectively, repeating the additions as a check. (7) Transcribe field notes and adjusted latitudes and departures, and verify transcript. (8) Calculate the meridian distances of the several stations and lines. (9) Calculate the latitude coordinates. (10) Calculate the partial trapezoidal areas by multiplying the meridian distances of the lines by the respective latitudes, pre-

Oct. 17, '99. Computer. J. Doe.
Data from pp. Transcript O.K.

COMPASS TRAVERSE OF FIELD A-B-C-D-E LATITUDES AND DEPARTURES.

Line	Adjusted Bearing	Observed Distance Ft.	Computation of Latitudes. Multiplication. (Lat. = Dist. × Cos. Bg.)	Logarithms.	Computed Latitude. Ft.	Lat. Cor. Ft.	Adjusted Latitude Ft.	Computation of Departures. Multiplication. (Dep. = Dist × Sin. Bg.)	Logarithms	Comp'd Dep'ture	Dep. Cor. Ft.	Adjusted Dep'ture.
AB	S32°45'E	336.5	336.5 4074.0 2692 0 1346 34 1 283.01	2.52698 9.92481 2.45180 (283.01)	S 283.0	-0.2	S 282.8	336.5 73043.0 16825 1346 30 2 182.03	2.52698 9.73316 2.26016 (182.04)	E182.0	0.0	E182.0
BC	S43°05'E	464.6	464.6 63037.0 32522 1394 14 2 339.32	2.66708 9.86353 2.53081 (339.32)	S 339.3	-0.2	S 339.1	464.6 60386.0 27876 3717 139 3 317.35	2.66708 9.83446 2.50154 (317.35)	E317.4	+0.1	E317.5
CD	S81°50'W	483.3	483.3 50241.0 4833 1933 97 2 68.65	2.68422 9.15245 2.83667 (68.65)	S 68.6	0.0	S 68.6	483.3 69893.0 43497 3866 435 39 3 478.40	2.68422 9.99567 2.67979 (470.40)	W478.4	-0.2	W478.2
DE	N22°05'W	616.0	616.0 46629.0 55440 1232 370 37 2 570.81	2.78958 9.96691 2.75649 (570.81)	N570.8	+0.3	N571.1	616.0 59573.0 18480 4312 308 55 3 231.38	2.78958 9.57514 2.36472 (231.59	W231.6	-0.1	W231.5
EA	N60°25'E	241.6	241.6 98334.0 9064 2174 72 15 2 119.27	2.38310 9.69345 2.07655 (119.28)	N119.3	+0.1	N119.4	241.6 49900.0 19328 1430 217 14 1 210.10	2.38310 9.93934 2.32244 (210.11)	E210.1	+0.1	E210.2
	P =	2142.0			N690.1	+0.4	N680.5			E709.5	+0.2	E709.7
					S690.9	-0.4	S690.5			W710.0	-0.3	W709.7
					S 0.8	0.8	0.0			W 0.5	0.5	0.0

Distribution of Error.

Line	Lat.		Dep.	
AB	3	0.2	2	0.0
BC	3	0.2	3	0.1
CD	1	0.0	5	0.2
DE	6	0.3	2	0.1
EA	1	0.1	2	0.1
	14	0.8	14	0.5

Error of Closure. (See Diagrams.)

Actual Error $= \sqrt{L^2 + D^2} = \sqrt{0.8^2 + 0.5^2} = 0.9$ ft.

Permissible Error $= \dfrac{3 \times P}{10000} = \dfrac{3 \times 5 \times 2142}{10000} = 3.2$ ft.

Oct. 17, '99. Computer. J. Doe.
Data from pp. Transcript O.K.

COMPUTATION OF AREA OF FIELD A-B-C-D-E, COMPASS TRAVERSE.

Station	Line	Adjusted Bearing.	Observed Distance Ft.	Adjusted Latitude. N. Ft.	S. Ft.	Adjusted Departure E. Ft.	W. Ft.	Meridian Station Ft.	Distance Line Ft.	Latitude Coördinate Ft.	Areas. N. Sq. Ft.	S. Sq. Ft.
A								E210.2		N119.4		
	AB	S32°45'E	336.5		282.8	182.0		(Mean=)	E301.2			85179
B								E392.2		S162.4		
	BC	S43°05'E	464.6		339.1	317.5			E551.0			186844
C								E709.7		S501.5		
	CD	S81°50'W	483.3		68.6		478.2		E470.6			32283
D								E231.5		S571.1		
	DE	N22°05'W	616.0	571.1			231.5		E115.8		66133	
E								0.0		0.0		
	EA	N60°25'E	241.6	119.4		210.2			E105.0		12549	
				690.5 =	690.5	709.7 =	709.7	1543.6 =	1543.6		78682	304306

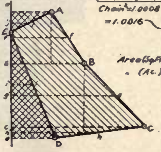
AUXILIARY CALCULATIONS.

Sta.	Line	Mer. Dist.	Lat. Coörd.
E		0.0	0.0
	EB	E210.2	N119.4
A		E210.2	N119.4
	AB	E182.0	S282.8
B		E392.2	S162.4
	BC	E317.5	S339.1
C		E709.7	S501.5
	CD	W478.2	S 68.6
D		E231.5	S571.1
	DE	W231.5	N571.1
E		0.0	0.0

Area Multiplications

AB	CD	EA
301.2 8282 60240 24096 602 241 85179	470.6 686 28236 3765 282 32283	105.1 4911 10510 1051 946 42 12549

BC	DE
551.0 1.933 165300 16530 4959 55 186844	115.8 1.175 57900 8108 116 11 66133



Chain² = 1.0008²
= 1.0016

304306
78682
225624
.61001
225624
226
135
Area (Sq. Ft.) = 225985
" (Ac.) = 5.188

serving consistent accuracy, and observing algebraic signs. (11) Determine the area by taking the algebraic sum of the partial areas. Reduce to acres, and correct for standard. Follow the prescribed form.

PROBLEM B5. ADJUSTMENT OF THE COMPASS.

(a) *Equipment*—Surveyors' compass, adjusting pin, small screw driver.

(b) *Problem.*—Make the necessary tests and adjustments of the surveyors' compass.

(c) *Methods.*—Observe the following program: (1) test the magnetism of the needle; (2) test the metal of the compass box; (3) test and adjust the plate levels; (4) test the sights; (5) test the pivot; (6) test the needle.

PROBLEM B6. COMPARISON OF DIFFERENT MAKES AND TYPES OF COMPASSES.

(a) *Equipment.*—Department equipment, catalogs of representative makers of compasses.

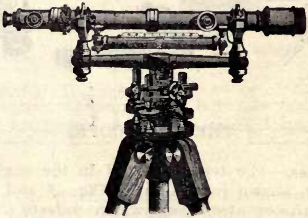
(b) *Problem.*—Make a critical comparison of the several types of compasses.

(c) *Methods.*—Examine the department equipment and study the several catalogs carefully, noting the characteristic features, prices, etc. The following items, at least, should be included in the tabulated report: name of instrument, length of needle, length of alidade, vernier, tripod, weight, price, etc.

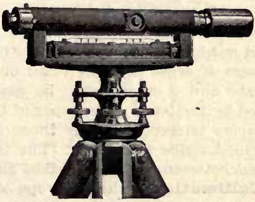
CHAPTER IV.

THE LEVEL.

Description.—The engineers' level consists of a line of sight attached to a bubble vial and a vertical axis. Two types of level, the wye and dumpy, Fig. 14, are used by engineers. In the former the telescope rests in Y-shaped supports, from which it may be removed. In the dumpy level the telescope is fixed. The dumpy is a favorite with British engineers and the wye level with Americans. The two types differ chiefly in the methods of adjustment. A third type, not shown in the cuts, is called the level of precision because of its use solely for work of extreme refinement.



ENGINEERS' WYE LEVEL.



DUMPY LEVEL.

Fig. 14.

In Fig. 15 are shown: (a) an architects' or builders' level of the wye type; (b) a roadbuilders level of the dumpy type; (c) a reconnaissance level with a decimal scale for reading horizontal distances direct; (d) a water level sometimes used in locating contours; (e) a Locke hand level; (f) a clinometer; (g) a binocular hand level.

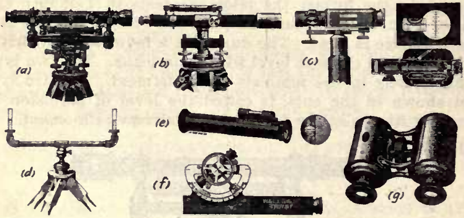


Fig. 15.

THE TELESCOPE.

Principles.—The telescope used in the engineers' level and transit, shown in section in Fig. 16 and 22, consists of an *objective* or *object glass* which collects the light and forms an image in the plane of the cross-hairs, and an *ocular* or *eyepiece* which magnifies the image and cross-hairs. The cross-hairs are thus at the common focus of the objective and eyepiece. The principle of this type of telescope, both optically and mechanically, may be illustrated by the photographic camera if cross lines be ruled on the ground glass focusing plate and a microscope be used in viewing the image formed by the lens. Telescopes of the above class are called *measuring* telescopes, while those of the opera glass type are termed *seeing* telescopes. The latter have no real image formed between the object glass and eyepiece.

Line of Collimation.—The telescope of the level or transit may be represented by a line, called the *line of collimation*, which joins the optical center of the objective and the

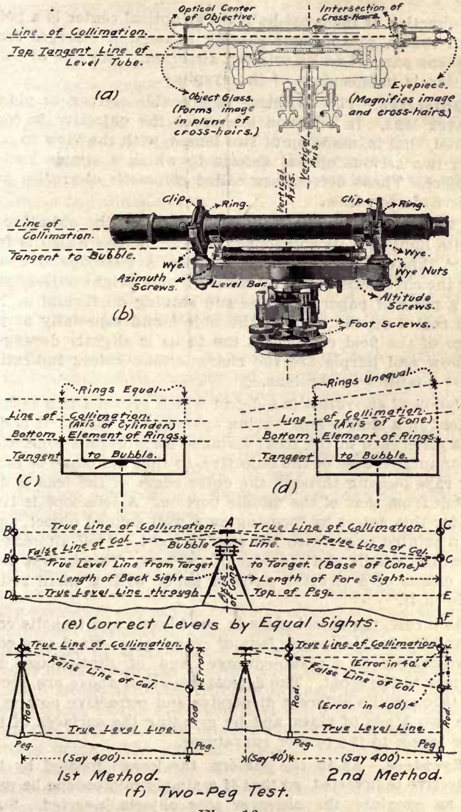

Optical Center of Objective.
Intersection of Cross-Hairs.
Line of Collimation.
Top Tangent Line of Level Tube.
(a)
Object Glass. (Forms image in plane of cross-hairs.)
Eyepiece. (Magnifies image and cross-hairs.)
Vertical Axis.
Vertical Axis.
Clip.
Ring.
Clip.
Ring.
Line of Collimation.
Tangent to Bubble.
Wye.
Wye.
Wye Nuts
Azimuth Screws.
Level Bar.
Altitude Screws.
(b)
Foot Screws.
Rings Equal
Line of Collimation. (Axis of Cylinder.)
Bottom Element of Rings.
Tangent to Bubble.
(c)
Rings Unequal
Line of Collimation (Axis of Cone)
Bottom Element of Rings.
Tangent to Bubble.
(d)
True Line of Collimation.
A
True Line of Collimation
False Line of Col.
Bubble Line.
False Line of Col.
True Level Line from Target to Target. (Base of Cone.)
Length of Back Sight
Length of Fore Sight
Axis of Cone
True Level Line through Top of Peg.
B
B'
C
C'
D
E
F
(e) Correct Levels by Equal Sights.
True Line of Collimation.
False Line of Col.
Error
Rod
Rod
True Level Line.
Peg.
Peg.
(Say 400')
1st Method.
True Line of Collimation.
(Error in 40')
False Line of Col.
(Error in 400')
Rod.
Rod.
True Level Line.
Peg.
Peg
(Say 40')
(Say 400')
2nd Method.
(f) Two-Peg Test.

Fig. 16.

intersection of the cross-hairs. The optical center is a point such that a ray of light passing through it emerges from the lens parallel to its original direction. The line of collimation is independent of the eyepiece.

Objective.—The objective is a double convex or plano-convex lens. In all good telescopes the objective is compound, that is, made up of two lenses, with the view to correct two serious optical defects to which a simple lens is subject. These defects are called *chromatic aberration* and *spherical aberration*.

Chromatic aberration is the separation by the objective of white light into its component colors. A lens which is free from this defect is called *achromatic*. A telescope is tested for the chromatic defect by focusing on a bright object, such as a piece of paper with the sun shining on it, and noting the colors on the edge of the object and especially at the edge of the field of view as the focus is slightly deranged. Yellow and purple are the characteristic colors indicating good qualities in the lens.

Spherical aberration is a defect which prevails to a serious extent in a simple lens having spherical surfaces. It is due to a difference in the focal distance for different concentric or annular spaces of the objective, so that the plane of focus for rays passing through the outer edges of the lens is different from that of the middle portion. A telescope is tested for this defect by focusing on a well defined object, such as a printed page, with the rays of light cut off alternately from the middle and the edge of the lens. This is best done by means of a circular piece of paper with a small round hole in it.

As a rule, the object glass in good levels and transits consists of a double convex lens of crown glass fitted to a concavo-convex or a plano-concave lens of flint glass, the former to the front. The defects described above are avoided through the different dispersive and refractive powers of the two kinds of glass, and by grinding the surfaces of the two lenses to the proper curvatures.

Eyepiece.—As in the camera, the image formed by the objective is inverted, so that if a simple microscope be used as an eyepiece, the observer sees objects inverted. Such an eyepiece is commonly used on the dumpy level, as shown in Fig. 14. This form of eyepiece consists of two plano-convex lenses with their convex sides facing each other. The form of eyepiece most used in American instruments is the erecting eyepiece in which two plano-convex lenses replace each of the two in the simpler form. The erecting eyepiece is much longer than the simple one, as may be seen at a glance in Fig. 14. While the simple eyepiece causes a little confusion at first, owing to the inversion of objects, it is much superior to the erecting eyepiece in the matter of clearness and illumination.

The chief inherent defect in the eyepiece is a *lack of flatness* of the field. A single lens usually causes a distortion or curving of straight lines in the image, especially towards the edge of the field. A telescope is tested for this defect by observing a series of parallel right lines, preferably a series of concentric squares, which fill the entire field of view.

In the best achromatic eyepieces, one or more of the separate lenses may be compounded, the curvatures being such as to eliminate the color defect and give rectilinear qualities to the lens or combination of lenses.

Definition.—The definition of a telescope depends upon the finish and also the accuracy of the grinding of the curved surfaces of the lenses. It may be tested by reading the time on a watch or a finely printed page at some distance from the instrument.

Illumination.—Illumination and definition are apt to be confused. Poor definition causes indefinite details, while poor illumination causes faintness in the image. The latter may be tested about dusk, or in a room which can be gradually darkened, and can be best appreciated if two telescopes of different illuminating qualities be compared.

Aperture of Objective.—The aperture or effective diameter of the objective is determined by moving the end of a pencil slowly into the field and noting the point where it first appears to the eye when held say 8 or 10 inches back from the eyepiece. The process should be repeated in the reverse order. The annular space is deducted from the actual diameter to obtain the real aperture.

Size of Field.—The field of the telescope is determined by noting the angle between the extreme rays of light which enter the effective aperture of the objective. With the transit telescope, the limiting points may be marked on the side of a building and the angle measured directly with the plates; or with either level or transit the angle may be calculated from the measured spread in a given distance. For simplicity, a distance of 57.3 feet may be taken, and the result reduced to minutes.

Magnifying Power.—The magnifying power of a telescope is expressed in diameters, or as the multiplication of linear dimension. It is determined most readily by making an observation with both eyes open, one looking through the telescope and the other by natural vision. The comparison may be made by means of a leveling rod, or the courses of brick or weather-boarding on the side of a house may be used in like manner.

Parallax.—Parallax is the apparent movement of the cross-hairs on the object with a slight movement of the eye, and is due to imperfect focusing of the eyepiece on the cross-hairs before focusing the objective. The eyepiece should be focused *with the eye normal*, the cross-hairs being illuminated by holding the note book page or other white object a few inches in front of the objective.

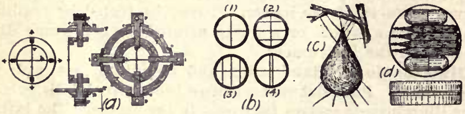


Fig. 17.

Cross-Hairs.—The cross-hairs are attached to a ring or reticule which is held by two pairs of capstan headed screws. The hairs usually consist of spider lines, although some makers use platinum wires for the purpose. To remove the reticule the eyepiece is taken out, one pair of screws is removed and a sharpened stick is inserted in a screw hole. The best spider lines are obtained from the spider's egg nest.

In Fig. 17, (a) shows the usual arrangement of the cross-hair ring and the method of attaching the hairs; (b) shows the number and positions of hairs used, (1) being the most common, (2) the form for stadia work with the transit and also for estimating the lengths of sights with the level, (3) a form used by some makers with the level, and (4) a style found in English levels; (c) shows the egg pod or case of the large brown spider (about half size) which yields the best lines for engineering instruments; (d) illustrates a convenient vest pocket outfit for replacing cross-hairs in the field, consisting of a supply of spider lines and some adhesive paper (bank note repair paper) each in a capsule or tin tube, and several sharpened sticks for stretching the hairs. Cross-hairs stretched in this manner may last indefinitely, or they may be fastened on permanently with shellac at the first opportunity.

THE BUBBLE VIAL.

Principle.—The spirit level consists of a sealed glass tube nearly filled with ether or other liquid, and bent or ground so that the action of gravity on the liquid may indicate a level line by means of the bubble. The delicacy of the bubble depends upon the radius of the curvature in a vertical plane, the greater the radius the more delicate the level. Thus, for example, a perfectly straight tube could not be used as a level.

Curvature of Bubble Vials.—Good bubble vials are now made by grinding or polishing the interior surface of a selected glass tube by revolution, as indicated in exaggerated form at (a) Fig. 18. As a general rule, only one side of the vial is actually used, it being customary to encase it in a brass tube having a slot or race on one side. However, both sides of the vial may be utilized, as in (b) and (c), Fig. 18, which show the *reversion level* adapted to the transit and wye level, respectively. Bubble vials of several sizes are shown in (d), Fig. 18. It was formerly customary to grind out only a portion of the upper side of the glass tube, as shown at (e). The cheap vial, consisting merely of a bent tube, used mostly in carpenters' and masons' levels, is shown at (f); and a method of increasing the precision of the bent tube by tilting it is indicated at (g) Fig. 18.

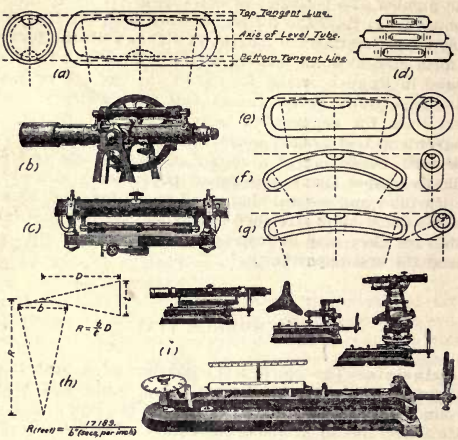


Fig. 18.

Delicacy.—The delicacy of the bubble vial is designated either by the radius, usually in feet, or by the central angle in seconds corresponding to one division or one inch of the bubble scale. Two methods are employed to determine the delicacy of level vials, (1) by the optical method, as at (h), Fig. 18, where the radius is calculated from an observed target movement at a given distance for an observed bubble movement, the two triangles being similar; and (2) by the level tester, as at (i), by means of which the angular movement is read from the micrometer head for a given movement of the bubble. The engineer usually employs the radial designation, while the maker expresses the delicacy in angular units. As shown at (h) and (i), Fig. 18, the radius in feet is equal to 17,189 divided by seconds per inch of bubble.

Bubble Line.—The relations of the bubble to the other parts of the instrument are best understood by representing the vial by a line. This line may be either the axis of the surface of revolution in (a), Fig. 18, or to provide for either of the three forms of vial shown, it may be taken as the tangent line at the middle or top point. This tangent line will be meant hereafter in referring to the bubble line.

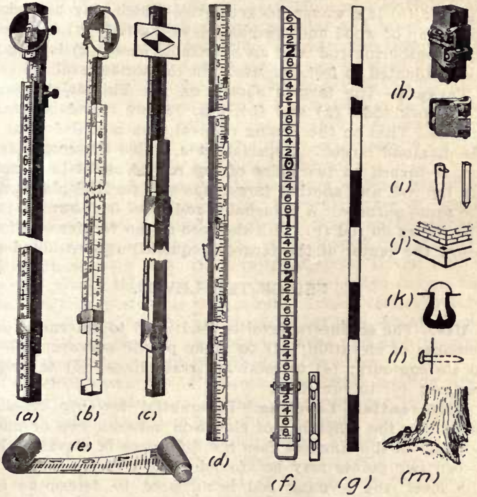


Fig. 19.

LEVELING RODS.

Types.—There are two classes or types of leveling rods; (1) *target* rods, having a sliding target which is brought into the line of sight by signals from the leveler; and (2) *self-reading* or speaking rods which are read directly by the leveler.

In Fig. 19, (a) is the Philadelphia rod; (b) the New York rod; and (c) the Boston rod. The first is either a target or self-reading rod; the second is a target rod, but may be read from the instrument when the rod is "short"; the Boston rod is strictly a target rod. The Philadelphia rod is perhaps the favorite for most purposes, and the Boston rod is used least. A folding self-reading rod is shown at (d), Fig. 19; (e) is a woven pocket device which may be tacked to a strip of wood and used as a leveling rod; (f) is a railroad contouring rod with an adjustable base; (g) is a plain rod graduated to feet, for use with the water level.

Targets.—The targets shown on the Philadelphia and New York rods, (a) and (b), Fig. 19, are called quadrant targets. That on the Boston rod, (c), is a modified form of the diamond target. A special form, called the corner target, is turned on two sides of the rod to assist in plumbing the rod, and another target has two parallel planes for the same purpose. A detachable rod level is shown at (h). The target on rod (b), with the zero of the vernier 0.09 foot below the center of the target, frequently causes blunders.

USE OF THE LEVEL.

Use.—The engineers' level is used: (1) to determine differences of elevation; (2) to make profile surveys; (3) to locate contours; (4) to establish grade lines; (5) to cross-section; (6) to run lines.

Differential Leveling.—Differential leveling consists of finding the difference of elevation between two or more points. In the simplest case the difference of elevation between two points may be found from a single setting of the level, the leveling rod being used to determine the vertical distance from the plane of the instrument to each of the two points, and the difference between the rod readings taken. When the distance between the two points is too great, either vertically or horizontally, or both, to admit of this simple process, two or more settings of the level are taken so as to secure a connected series of rod readings, the algebraic sum of which gives the desired difference of elevation. This difference may be expressed either by the numerical result of the algebraic sum of the rod readings, or by assuming an elevation for the beginning point and calculating the elevation of the closing point by means of the observed rod readings.

A back sight is a rod reading taken to determine *the height of the instrument.* A *fore sight* is a rod reading taken to determine *the height of a point.* A *bench mark* is a point selected or established for *permanent reference* in leveling operations. A *turning point* is a *temporary* reference point used in moving the instrument ahead to a new setting. The same point is often both a turning point and bench mark. The *datum* is the plane or surface of reference from which the elevations are reckoned; it may be sea level, or an arbitrary local datum. A *level* line is a line parallel to the surface of a smooth body of water. A *horizontal* line is tangent to a level line at any point. The curvature varies as the square of the distance from the point of tangency, and is 0.001 foot in 204 feet, or 8 inches in one mile.

In Fig. 19, (i) shows a metal and also a wooden peg commonly used for turning points. Several forms of bench marks are shown in Fig. 19; (j) is a mark on the corner of a stone water-table; (k) a rivet leaded into a hole drilled in a stone slab, (l) a railroad spike driven into a wooden post or telegraph pole; (m) a projection cut on the root of a tree, preferably with a spike driven vertically into the top of the bench, and usually with a blaze above marked "B. M. No.—." All bench marks and also turning points should be clearly described in the notes.

Two chief essentials in correct differential leveling are: (1) that *the bubble be in exactly the same position* (usually the middle) *on both back and fore sight*; and (2) that *the length of back sight and fore sight*, horizontally, *shall be balanced.* It is seen at (e), Fig. 16, that with the bubble always in the middle, the line of collimation generates a *horizontal plane* when in perfect adjustment, but a *cone* with *axis vertical* when out of adjustment; so that in taking equal distances in the opposite directions, the *base* of the *cone* is used, this base being parallel to the true collimation plane. In the best leveling practice *the instrument is adjusted as perfectly as possible* and then used so that *the residual errors balance each other.*

The three common styles of leveling rod may be read to 0.001 foot by vernier or by estimation on a scale of 0.005 foot. However, for most kinds of leveling, it is an absurd refinement to read the rod closer than 0.01 foot, especially with the usual maximum length of sight of 350 to 400 feet, and with the more or less sluggish bubbles supplied in the general run of leveling instruments. Furthermore, the horizontal hair usually covers 0.01 foot or so of the target at the maximum length of sight, that is, the target can move that amount without being noticed by the observer.

Profile Leveling.—Profile leveling consists of finding the relative elevations of a series of representative points along a surveyed line, for the purpose of constructing a profile or vertical section. The skeleton of profile leveling, that is, the precise bench marks and turning points with the successive heights of instrument, is identical with differential leveling, already described. Having determined the height of instrument by taking a back sight on a bench mark of known or assumed elevation, rod readings are taken at proper intervals along the measured and staked line. These readings are fore sights, but they are usually termed *intermediate sights* to distinguish them from the more precise rod readings taken on turning points and bench marks. On railroad surveys intermediate sights are taken usually to the nearest 0.1 foot on the ground; but in other cases, such as tile and sewer surveys, intermediates are often read to the nearest 0.01 foot on small pegs driven beside the station stakes flush with the surface of the ground. In railroad work, the benches, turning points, and intermediates of special importance are commonly read to 0.01 foot, although some engineers persist in the questionable practice of taking the nearest 0.001. In drainage surveys the nearest 0.01 foot is usually taken on bench marks, although more carefully than on the intermediate peg points, and the nearest 0.1 foot is read on ground points.

The errors of profile leveling are balanced on turning points by equal back and fore sights, as in differential leveling. If the instrument is seriously out of adjustment, an error is made in the case of odd bench marks with unbalanced sights, and also on all intermediate sights. However, the error is usually unimportant when ground readings are taken to the nearest 0.1 foot. In important leveling, such as canal and drainage work, it is customary to run a line of check levels to prove the benches, before construction begins.

The profile is plotted to an exaggerated scale vertically on a special paper, called profile paper. Three kinds, known as plates A, B and C, are in general use. The most common is plate A, which is ruled in ¼-inch squares with a further subdivision to 1-20 inch vertically. In railroad profiles the scales most used are 400 feet to the inch horizontally and 20 feet vertically. A still greater exaggeration is generally used in drainage profiles.

Contour Leveling.—Contour leveling is an application of the methods of profile leveling to the location of contour lines, that is, lines having the same elevation. Two methods are employed: either (1) actually establishing points on the adopted contour planes on the ground and then locating these points; or (2) taking random elevations at representative points and interpolating the contour lines from the plotted data. The latter is the more common. The chief purpose of contour leveling is to make a contour map, and the process is essentially a part of topographic surveying, where it will be more fully considered.

Grade Lines.—The establishment of grade lines is usually the concluding part of profile leveling. After making the profile, the grade line is established by stretching a fine thread through the ruling points, taking into account the controlling conditions, such as maximum gradient or earthwork quantities on a railroad profile, the carrying capacity or the scour in the case of a ditch, etc. After laying the grade line on the profile, notes are made of the data, and the actual grade line is established. Two methods are used: (1) the height of instrument is determined as usual, and stakes are driven at measured intervals with their tops to match calculated rod readings; and (2) a limited number of ruling points are established by the first method or otherwise, and the remaining stakes are "shot in" by constructing a line parallel to the ruling line used. The latter is more rapid, since a constant rod reading is used; however, the method is unreliable unless the fore sight be checked frequently on a fixed target.

Cross-Sectioning.—Cross-sectioning consists of staking out the limits of the transverse section of an excavation or embankment for the purpose of construction, and usually includes the collection of data for the calculation of the quantities. This may be done either with the engineers' level, rod and tape line, or with special rods called cross-section rods. The notes are taken as rectangular coordinates, usually with reference to the center of the finished roadbed. The slope stakes are set where the side slope lines pierce the surface of the ground.

Running Lines.—Lines are sometimes run with the engineers' level, provision being made in most good levels for the attachment of a plumb bob. A line may be prolonged by sighting in two points ahead. A clamp and tangent movement are necessary. Some builders' levels have a needle and also a roughly divided horizontal circle for use in staking out buildings.

Practical Hints.—The following practical suggestions apply more or less directly to all kinds of leveling, and also in a general sense to transit work.

Speed.—Cultivate the habit of briskness in all the details of the work. While undue haste lowers the standard of the results, an effort should be made to gain speed steadily without sacrificing precision. Gain time for the more important details by moving rapidly from point to point. On rapid surveys both leveler and rodman often move in a trot. Neither rodman nor leveler should delay the other needlessly.

Care of Instruments.—Do not carry the level on the shoulder in climbing fences. Clamp the telescope slightly when hanging down. Keep the tripod legs at the proper tightness, and avoid looseness in the tripod shoes. Avoid undue exposure to the elements, and guard the level from injury. Do not leave the instrument standing on the tripod in a room over night.

Setting Up.—In choosing a place to set the level up, consider visibility and elevation of back point and probable fore sight. Set up with plates about level. On side-hill ground place one leg up hill. In general, place two tripod shoes parallel to the general line of the levels.

Leveling Up.—A pair of foot screws should be shifted to the general direction of the back or fore sight before leveling up. Set the foot screws up just to a snug bearing and no tighter. If either pair of screws binds, loosen the other pair a little. The bubble moves with the left thumb. Level up more precisely in the direction of the sight than transverse to it, but do not neglect the latter. Inspect the bubble squarely to avoid parallax, and also to prevent such blunders as reading the bubble five spaces off center.

Observations.—Adjust the eyepiece for parallax with the eye unstrained. It is much easier on the eyes to observe with both eyes open. Read at the intersection of the cross-hairs, since the horizontal hair may be inclined. Set the target approximately, check the bubble, and repeat the process several times before approving the sight. Be certain that the bubble is exactly in the middle at the instant of approving the target. If the level has horizontal stadia lines, beware of reading the wrong hair (the reticule may be rotated one-quarter so as to have the extra hairs vertical, or a filament may be attached to the middle horizontal hair to assist in identifying it). Avoid disturbance of the tripod by stepping about the instrument. Assist the rodman in plumbing the rod. Let signals be perfectly definite both as to direction and amount, using the left hand for "up" and the right for "down", or vice versa.

The leveler can work much more intelligently if he knows the space covered on the rod by one division of the bubble scale at the maximum length of sight, and also the space on the rod hidden by the cross-hair.

Balancing Sights.—Balance the length of back sight and fore sight, and record the approximate distances. The distances in the two directions may be made equal roughly by equality of focus, but it is better on careful work to pace the distances or determine them by means of the stadia lines in the level. If necessary to unbalance the sights, they should be balanced up at the first opportunity, and in general they should be in balance when closing on important benches. When leveling up or down steep slopes, follow a zigzag course to avoid short sights. Take no sights longer than 350 or 400 feet.

Leveling Rod.—The rod should be carefully plumbed, to accomplish which the rodman should stand squarely behind the rod and support it symmetrically between the tips of the extended fingers of the two hands. With "short" rods avoid the somewhat common blunder of 0.09 foot when the vernier slot is below the center of the target. With "long" rods, see that the target has not slipped from its true setting before reading the rod. Read the rod at least twice, and avoid blunders of 1 foot, 0.1 foot, etc. Careless rodmen sometimes invert the rod. Each rod reading on turning points and bench marks should, when practicable, be read independently by both rodman and leveler.

Bench Marks and Turning Points.—Wooden pegs or other substantial points should be used to turn the instrument on. Select bench marks with reference to ease of identification, the balancing of sights, freedom from disturbance, etc. As a rule, each bench mark should be used as a turning point so that the final closure of the circuit may prove the bench.

Record and Calculations.—Describe bench marks and turning points clearly. It is good practice to apply algebraic signs to the back and fore sight rod readings. The elevations should be calculated as fast as the rod readings are taken, and calculations on turning points should be made independently by leveler and rodman, and results compared at each point. The rodman may keep turning point notes in the form of a single column. The calculations should be further verified by adding up the columns of back sights and fore sights for each circuit, or page, or day's work, and the algebraic sum of the two compared with the difference between the initial and last calculated elevation.

Error of Closure.—A circuit of levels run with a good level by careful men, observing all the foregoing precautions, should check within 0.05 foot into the square root of the length of the circuit in miles (equivalent to 0.007 foot into the square root of the length of the circuit in 100-foot stations.) In closing a circuit, the error should be carefully determined, as above indicated, and the value of the coefficient of precision found. (See discussion of errors of leveling and precision diagrams in the chapter on errors of surveying.)

ADJUSTMENT OF THE WYE LEVEL.

Elementary Lines.—The principal elementary lines of the wye level, as shown in Fig. 16, are: (1) the line of collimation; (2) the bubble line; (3) the vertical axis. For the purpose of adjustment there should be added to these: (4) the axis of the rings; (5) the bottom element of the rings. The following relations should exist between these lines; (a) the line of collimation and bubble line should be parallel; (b) the bubble line should be perpendicular to the vertical axis. The first of these relations involves two steps, viz., (1) to make the bubble line parallel to the bottom element of the rings, and (2) to make the line of collimation coincide with the axis of the rings. The other relation involves the wye adjustment, and is similar to the plate level adjustment described in the chapter on the compass.

Bubble.—*To make the bubble line parallel to the bottom element of the rings.*—Two steps are involved, (a) to place the bubble line *in the same plane* with the bottom element, and (b) to make the two lines *parallel.*

Azimuth Screws.—To make the bubble line in the same plane with the bottom element of the rings.—Clamp the level over a pair of foot screws, loosen the wye clips, and level up; rotate the telescope through a small angle, and if the bubble moves away from the middle, bring it back by means of the *azimuth* adjusting screws. Test by rotating in the opposite direction. Leave the screws snug.

Altitude Screws—To make the bnbble line and the bottom element of the rings parallel.—Make the element level with the foot screws and bring the bubble to the middle by means of the *altitude* adjusting screws. The element is made level by the method of reversions as follows: With the level clamped over a pair of foot screws, as above, lift the clips and level up precisely; cautiously lift the telescope out of the wyes, turn it end for end, and *very gently* replace it in the wyes; if the bubble moves, bring it half way back by means of the *foot screws.* Before disturbing adjusting screws make several reversals, and conclude the adjustment with screws snug. This end for end reversal is similar to that made with the carpenter's level, the straight edge of the level corresponding to the element of the rings. The lines involved are shown in Fig. 16.

Line of Collimation.—*To make the line of collimation coincide with the axis of the rings.*—Loosen clips, sight on a point, say a nail head or the level target, more distant than the longest sight used in leveling; rotate the telescope half way and note the movement of the hair, if any. The line of collimation generates a cone, the axis of which is that of the rings, and the apex of which is at the optical center of the objective. Correct one-half the observed error by means of the capstan headed screws which hold the cross-hair ring. Gradually perfect the adjustment until the intersection of the cross-hairs remains fixed on the same point when reversed by rotation with reference to either hair. The adjustment should be concluded with the screws at a snug bearing.

After collimating the instrument for a long distance, the adjustment should be checked for a short distance, say 50 or 100 feet, so as to test the motion of the optical center of the objective.

Rings.—*The theory of the wye level demands perfect equality of the rings*, that is, the parallelism of the axis and element, as in (c), Fig. 16. Should the rings be *unequal*, either from poor workmanship or uneven wear in service, they form a *cone* instead of a *cylinder*, and the axis is not parallel to the element, as in (d), Fig. 16. Under the latter conditions, the principle of the wye level fails, and an independent test is demanded. This is known as the two-peg test, the details of which are shown in (e) and (f), Fig. 16, and described in the adjustments of the dumpy level. If, after making the wye level adjustments above described, the two-peg test shows that the line of collimation and bubble line are not parallel, the rings are probably unequal and the instrument should thereafter be adjusted as a dumpy level. However, hasty conclusions should be guarded against.

In case the instrument has a reversion level, shown at (c), Fig. 18, the equality of the rings may be tested by first adjusting the top tangent line of the bubble vial parallel to the bottom element of the rings, and then after rotating the telescope half way round in the wyes, compare the bottom (now above) tangent line of the vial with the top (now below) element of the rings, all by the end for end reversion. However, the exact parallelism of the top and bottom tangent lines of the reversion level should first be proven by the two-peg method.

Wyes.—*To make bubble line perpendicular to the vertical axis.—Make the vertical axis vertical and bring the bubble to the middle by means of the wye nuts.* The vertical axis is made vertical by reversion thus: With clips pinned, level up; reverse over the same pair of screws, and bring the bubble half way back with the foot screws. When adjusted, the bubble will remain in the middle during a complete revolution. This adjustment is identical in principle with the plate level adjustment of the compass and transit, illustrated in (a), Fig. 13. The wye adjustment should follow the adjustment of the bubble line parallel to the element of the rings. The wye adjustment is a convenience, not a necessity.

Centering the Eyepiece.—After collimating the level, the cross-hairs should appear in the center of the field. The eyepiece is centered by moving its ring held by four screws. This adjustment is desirable, but not essential.

ADJUSTMENT OF THE DUMPY LEVEL.

Elementary Lines.—The principal elementary lines of the dumpy level are identical with those of the wye level: (1) the line of collimation; (2) the bubble line; (3) the vertical axis. As in the wye level, the bubble line should be (1) perpendicular to the vertical axis, and (2) parallel to the line of collimation. However, owing to the difference in the construction of the two types of instrument, the auxiliary elementary lines are not recognized in the dumpy level. The transit with its attached level is identical in principle with the dumpy level.

Bubble.—*To make the bubble line perpendicular to the vertical axis.—Make the vertical axis vertical by the method of reversions, and adjust the bubble to the middle.* This adjustment is identical in principle with the plate level adjustment, shown in (a), Fig. 13. The bubble should remain in the middle through a complete revolution.

Line of Collimation.—*To make the line of collimation parallel to the bubble line.—Construct a level line, and adjust the cross-hairs to agree with it.* The level line is determined either by using the surface of a pond of water, or by driving two pegs at equal distances in opposite directions from the instrument, and taking careful rod readings on them with the bubble precisely in the middle, as shown at (e), Fig. 16. For simplicity, the two pegs may be driven to the same level, or two spikes may be driven at the same level in the sides of two fence posts, say 400 feet apart. Otherwise, determine the precise difference of elevation, as indicated in (e), Fig. 16. Then set the level almost over one of the pegs, level up, and as in the first method of (f), Fig. 16, set the target of the leveling rod at the line of collimation, as indicated by the center of the object glass or eyepiece, (this can be done more precisely than most levels will set the target at 400 feet distance); now with the rod on the other peg, sight at the target (shifted to allow for the difference if the two pegs are not on the same level); adjust the cross-hair to the level line so constructed. If preferred, the second method shown in (f), Fig. 16, may be used; the level is set back of one peg, rod readings are taken on both pegs, allowance made for the difference in level of the two pegs, if any, the inclination of the line of collimation determined, correction made for the small triangle from the level to the first peg, and finally the level line constructed by means of the calculated rod readings. The second method is simplified and made practically equivalent to the first by setting the level at minimum focusing distance from the first peg. The small corrective triangle is thus practically eliminated. This process is called the two-peg adjustment.

The foregoing method ignores curvature of the earth (equal to 0.001 foot in about 200 feet, or 0.004 foot in 400 feet) which is less than the error of observation with most levels.

Uprights.—In some dumpy levels the uprights which connect the telescope with the level bar are adjustable, similar to the wyes of the wye level. This adjustment is designed to bring the bubble line perpendicular to the vertical axis in case the bubble is first adjusted parallel to the line of collimation. However, the best order is that already described, viz., first adjust the bubble line perpendicular to the vertical axis, and then the line of collimation parallel to the bubble line, in which case the adjustable uprights are unnecessary.

PROBLEMS WITH THE LEVEL.

PROBLEM C1. DIFFERENTIAL LEVELING WITH THE HAND LEVEL (OR WATER LEVEL.)

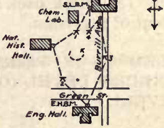
(a) *Equipment.*—Hand level (or water level), rod graduated to feet.

(b) *Problem.*—Run an assigned level circuit with the hand

Leveler, J. Doe. B. & B. Dumpy Level.
Rodman, R. Roe. N.Y. Rod, Locker 15.

LEVEL CIRCUIT, SEA-LEVEL B.M. TO ENG'S HALL B.M. 1 AND RETURN.

Sta.	B.S.	H.I.	F.S.	Elev.	Sights B.S.	Sights F.S.	Oct. 20, '99, (2 hours.) Clear and warm.
S.L.B.M.				720.00	180		Bolt, E. end, S. coping, E. entrance, University Hall.
$\overline{\wedge}$ (1)	+2.60	722.60				180	
⊙ 1			−7.56	715.04	200		W. end, N. coping, W. entrance, Natural Hist. Hall.
$\overline{\wedge}$ (2)	+2.00	717.04				210	
E.H.B.M.1			−3.42	713.62	280		Bolt, W. end, 1st step, S. entrance, Engineering Hall
$\overline{\wedge}$ (3)	+5.94	719.56				270	
⊙ 2			−0.30	719.26	80		Cement walk, 20' N.E., N. entrance, University Hall.
$\overline{\wedge}$ (4)	+3.87	723.13				80	
S.L.B.M.			−3.14	719.99			Bolt, starting point above described.
	+14.41		−14.42		740	740	
			+14.41	720.00	L = 14.8		Permissible error = 0.007 ft. $\sqrt{L}$ = 0.027 ft.
			− 0.01	−0.01			Actual error of closure = 0.01 ft.
			(Check.)				



H. H. Sherwin, Leveler.
C. E. Beyer, Rodman.

LEVELS FROM REYNOLDS TO LAFAYETTE, IND.

Sta.	B.S. +	H.I.	F.S. −	Elev.	Sights B.S.	Sights F.S.	Nov. 29, 1899. Cloudy, cool. B. & B. Dumpy Level.
B.M. 1	4.20	694.02		689.82	240		Penna R.R., B.M., M.P. 27, driven rail. (Levels
⊙ 1	5.95	696.30	3.67	690.35	165	240	Peg. direct from tide gage at
⊙ 2	5.70	698.18	3.82	692.48	240	240	" Sandy Hook, N.Y.)
⊙ 3	4.93	698.74	4.37	693.81	240	166	" (South along Monon R.R.)
⊙ 4	4.23	699.75	3.22	695.52	240	240	"
⊙ 5	4.24	699.04	4.95	694.80	240	240	"
⊙ 6	4.73	699.33	4.44	694.60	240	240	"
⊙ 7	4.44	699.61	4.16	695.17	300	240	" opposite Catholic Chch., Reynolds.
⊙ 8	5.18	699.39	5.40	694.21	300	300	"
⊙ 9	4.54	697.86	6.07	693.32	300	300	" E. side trk, betw. tel. poles 2289–90.
⊙ 10	4.81	698.12	4.55	693.31	300	300	"
⊙ 11	4.82	698.86	4.08	694.04	300	300	" betw. tel poles 2296–97, E. side trk.
B.M. 2			3.76	695.10		200	B.M. on oak, tel. pole 2299, 50' E. trk.
⊙ 12	4.10	698.00	4.96	693.90	300	300	Peg at tel. pole 2300.
⊙ 13	3.65	698.13	5.52	692.48	360	300	" " " " 2305.
⊙ 14	2.71	692.92	5.92	690.21	360	360	"
⊙ 15	3.21	690.03	6.10	686.82	360	360	" " " " 2313, E. side trk.
B.M. 3			3.15	686.88		150	N.E. cor. parapet wall, Bridge No. 97.4.
⊙ 16	4.65	690.83	3.85	686.18	360	360	Peg.
B.M. 4			5.29	685.54		100	Monon R.R., B.M. {Oak, 20' S. tel. pole 2320. 80' E. of trk. El. = 688.12}
⊙ 17	4.51	691.21	4.13	686.70	360	360	Peg.
⊙ 18	5.16	691.37	5.00	686.21	300	360	"
⊙ 19			5.13	686.24		300	" at tel. pole 2330.
	85.76		89.34	689.82	= B.M. 1		
			85.76	−3.58			
			−3.58	Check.			

level (or water level), observing the nearest 0.1 foot by estimation, and closing back on the starting point.

(c) *Methods.*—(1) Determine the correct position of the bubble of the hand level by sighting along a water table, or sill course of a building, or by the principles of the two-peg test. (If the water level is used, fill the tube so as to have a good exposure of the colored water in the glass uprights.) (2) Take sights of 100 feet or so (paced), estimating the rod reading to the nearest 0.1 foot; balance back and fore sights; assume the elevation of the starting point, and keep the notes in a single column by addition and subtraction. (3) Check back on the first point. Determine the coefficient of precision.

PROBLEM C2. DIFFERENTIAL LEVELING WITH ENGINEERS' LEVEL (OR TRANSIT WITH ATTACHED LEVEL).

(a) *Equipment.*—Engineers' level (or transit with attached level), leveling rod, hatchet, pegs, spikes.

(b) *Problem.*—Run the assigned level circuit, observing the nearest 0.01 foot, and closing back on the initial point.

(c) *Methods.*—Follow the practical suggestions given at the conclusion of the "Use of the Level," giving special attention to the following points: (1) eliminate parallax of the eyepiece; (2) balance back and fore sight distances; (3) have the bubble precisely in the middle at the instant of sighting; (4) both rodman and leveler read each rod and also make the calculations independently; (5) calculate elevations as rapidly as rod readings are obtained; (6) plumb the rod; (7) avoid blunders; (8) determine coefficient of precision; (9) no sights longer than 350 or 400 feet. Follow the first form shown to begin with,—the other after several circuits have been run.

PROBLEM C3. PROFILE LEVELING FOR A DRAIN.

(a) *Equipment.*—Engineers' leveling instrument, leveling rod, 100-foot steel tape, stakes, pegs, axe.

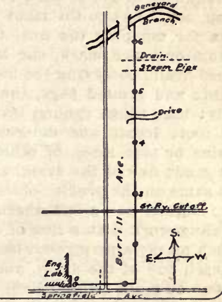
(b) *Problem.*—Make a survey, plat and profile, with estimate of cuts and quantities for a drain under assigned conditions.

Head Chainman, J. Doe.
Rear Chainman, R. Roe.

SURVEY FOR A DRAIN FROM ENGINEERING LABORATORY.

April 26, 1900. (2 hours) Cloudy and cool.
100-Ft. Steel Tape, No. 275, Locker No 35.

Sta.	Description.
+36	Bed of stream.
+32	Suitable outlet for drain, 11' W. of W. face of stone arch bridge.
+23	Break of N. bank, Boneyard Branch.
6	S. edge of 7' drive.
+60.5	Crosses drain from Conservatory.
5+42.3	Steam pipe line to Conservatory.
4+52	Center of 7' drive to Conservatory.
+68.6, +64.8	Rails of main track, U. & C. St. Ry. (Cutoff thro' University grounds.)
+58.6, +53.6	Rails of side track, Urbana & Champaign Electric Ry.
2	Drain is 2' L. (E.) of stake.
1+21.5	Turns S. in W. parking, Burrill Ave.
+73	12" Ash tree 6' to R.
+70, +64	Cement walk, E. side Burrill Ave.
+54.6	Crosses Military Hall steam pipe line.
0	A point 3' W. and 2' S. of N.W. Cor. of Eng. Lab. Line runs thence W. parallel to and 9' S. of S. line of Springfield Ave. Stakes are set 2' to R. of proposed trench for drain, with leveling pegs flush with ground beside stakes.

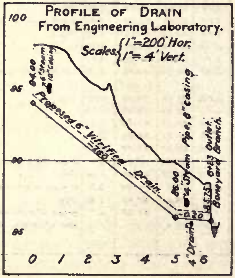


Leveler, R. Roe.
Rodman, J. Doe.
Fauth Wye Level.
Phila. Rod, Lkr. 20.

LEVEL NOTES FOR A DRAIN FROM ENGINEERING LABORATORY.

Sta.	B.S.	H.I.	F.S.	Elev.	Grade.	Cut.	Oct. 23, '99. (2 hrs.) Clear and cool.
				100.00			S. end stone sill, W. door, Eng. Lab.
⊼	+1.23	101.23					Station stakes are 2' R. proposed trench.
0			3.05	98.18	94.00	4.18	Peg driven flush with ground beside stake.
+50			3.22	98.01	93.20	4.81	"
+54.6			3.2	98.0	93.13	4.9	Ground, 6" steam pipe, 10" casing, top 2'.6 deep
+66			3.23	98.00	92.94	5.06	Cement walk, E. side Burrill Ave.
1			3.38	97.85	92.40	5.45	Peg.
+21.5			3.72	97.51	92.06	5.45	Drain turns S. in W. parking Burrill Ave.
+50			4.65	96.58	91.60	4.98	Peg.
2			5.71	95.52	90.80	4.72	"
+50			6.31	94.92	90.00	4.92	"
● +53			−5.85	95.38	89.86	5.52	(Turning Point.) N. rail, main track, U. & C. Ry.
⊼	+0.13	95.51					
3			2.08	93.43	89.20	4.23	Peg.
+50			3.45	92.06	88.40	3.66	"
4			4.34	91.17	87.60	3.57	"
+50			5.50	90.01	86.80	3.21	"
5			5.52	89.99	86.00	3.99	"
+42.3			6.1	89.4	85.91	3.5	Ground, 4" steam pipe 6" casing, top 2'.0 deep
+50			6.26	89.25	85.90	3.35	Peg.
+60.5			6.4	89.1	85.88	3.2	Ground, 4" vitrified drain, top 3'.4 deep.
6			7.11	88.40	85.80	2.60	Peg.
+23			7.0	88.5	85.75	2.7	Break of N. bank, Boneyard Branch.
+36			10.9	84.6			Bed of stream, water 10" deep.

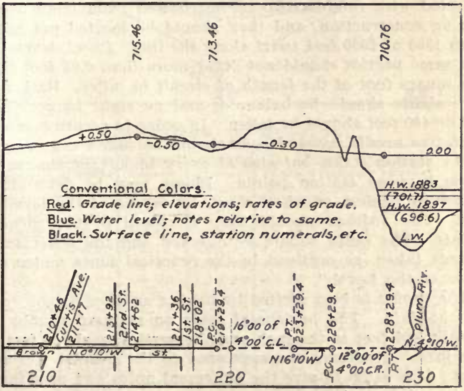
(c) *Methods.*—(1) Examine the ground, determine the head and outlet of the drain, and select the general route. (2) Stake out the line. set stakes every 50 feet, or oftener if required to get a good profile, and drive a ground peg flush, say 2 feet to the right (or left) of each stake; record data for mapping the line. (3) Starting with the assigned datum or bench mark, run levels over the line of the proposed drain, observing the nearest 0.01 foot both on turning points and ground pegs, the former somewhat more carefully; take rough ground levels, as required, to the nearest 0.1 foot; locate and determine the depth of intersecting drains or pipe lines, or other objects which may influence the grade line of the drain, and secure full data for placing the same on the profile; observe due care with the back and fore sights, as in differential leveling, and conclude the leveling work with a line of check levels back to the initial bench mark; a permanent bench mark should be established at each end of the drain, and if the length is considerable, at one or more intermediate points as well. (4) Make plat and profile of the drain line; lay the grade line, taking into account all ruling points; calculate the cuts, both to the nearest 0.01 foot, and also to the nearest ¼-inch; mark the latter on the stakes for the information of the ditcher, using waterproof keel and plain numerals; make estimate of the quantity of drain pipe, and of the cost of the job. Follow the accompanying forms.



(PROFILE LEVEL NOTES, GROUND ELEVATIONS TO 0.1 FOOT.)

S.	+	⊼	−	R.	E.	
209		718.33		5.0	713.3	In Brown St. (Unimproved.)
210				4.7	713.6	"
211				3.9	714.4	"
B.M.26	6.79	723.87	1.25		717.08	Water plug, N. bolt, N.W. cor., Brown-Curtis.
212				7.6	716.3	Ground, Brown St.
213				6.4	717.5	" "
214				5.9	718.0	" "
+50				4.0	719.9	" "
215				6.1	717.8	" "
216				8.0	715.9	" "
217				8.5	715.4	" "
218				10.3	713.6	" "
219				12.2	711.7	In corn field.
⊙Stake	9.22	721.64	11.45		712.42	⊙Stake Sta. 219.
220				8.6	713.0	Corn field.
221				4.4	717.2	"
222				2.7	718.9	"
223				2.9	718.7	"
224				2.3	719.3	"
225				3.4	718.2	Timber pasture
+90				12.4	709.2	Gully.
226				11.2	710.4	
+35				6.0	715.6	Break of bank, Plum River.
B.M.27	2.04	713.52	10.16		711.48	B.M., root, 24" elm, 72' R., Sta. 226+65.
+80				6.0	707.5	
	+18.05	718.33	−22.86			
		−4.81	+18.05			
			−4.81	Check.		

Column Headings { S = station. + = back sight. ⊼ = height inst. − = fore sight. R = rod (intermediate). E = elevation.



PROBLEM C4. RAILROAD PROFILE LEVELING.

(a) *Equipment.*—Engineers' leveling instrument, leveling rod, 100-foot steel tape, stakes, axe.

(b) *Problem.*—Run levels over a short section of line staked out after the manner of railroad surveys, for the purpose of constructing a profile.

(c) *Methods.*—Follow the general process outlined in the preceding problem, taking rod readings to the nearest 0.01 foot on turning points and bench marks, and also on important profiling points, when consistent; but take ground rod readings only to the nearest 0.1 foot. In calculating elevations, preserve the same degree of exactness in the result as observed in the rod reading, that is, when the rod readings are taken to the nearest 0.1 foot, use only the nearest 0.1 foot in the height of instrument to determine the elevations. When a hub or station stake is to be used as a turning point, the notes should show the ground rod and elevation to the nearest 0.1 foot on the line preceding the precise turning point record. Bench marks should be selected with reference to their freedom from disturbance during construction, and they should be located not more than 1500 or 2000 feet apart along the line. Check levels by the same parties should not differ more than 0.05 foot into the square foot of the length of circuit in miles. Back and fore sights should be balanced, and no sight longer than 350 or 400 feet should be taken. In order to secure a representative profile, ground rods should be taken not only at every station stake, but also at every important change of slope between station points. Pluses may be determined either by pacing, or when short, by means of the leveling rod. The rodman should keep a record of the turning points. The notes should be checked and the other safeguards taken, as outlined in the practical hints under the "Use of the Level."

The profile is best plotted by having another person read off the data. The horizontal scale on railroad profiles is usually 400 feet to the inch and the vertical scale 20 feet to the inch. Gradients are expressed to the nearest 0.01 per cent. It is usual to give the alinement notes and prominent topography, as shown.

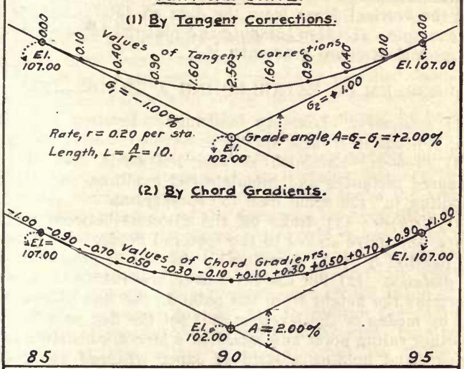
PROBLEM C5. VERTICAL CURVE.

(a) *Equipment.*—Drafting instruments, profile paper.

(b) *Problem.*—Connect two grade lines by a parabolic curve, as assigned.

(c) *Methods.*—(1) Plot the given grade lines, station num-

VERTICAL CURVE.



COMPARISON OF RESULTS.

Station.	Elevation of Grade Tangent.	By Tangent Corrections: Tangent Correction.	Curve. Elevation.	By Chord Gradients: Chord Diff.	Gradient.	Curve Elevation.
	Ft.	Ft.	Ft.	Per Cent.	Per Cent.	Ft.
84	108.00				(-1.00)	
85(P.C.)	107.00	+0.00	107.00	+0.10	-0.90	107.00
86	106.00	+0.10	106.10	+0.20	-0.70	106.10
87	105.00	+0.40	105.40	+0.20	-0.50	105.40
88	104.00	+0.90	104.90	+0.20	-0.30	104.90
89	103.00	+1.60	104.60	+0.20	-0.10	104.60
90 (Apex)	102.00	+2.50	104.50	+0.20	+0.10	104.50
91	103.00	+1.60	104.60	+0.20	+0.30	104.60
92	104.00	+0.90	104.90	+0.20	+0.50	104.90
93	105.00	+0.40	105.40	+0.20	+0.70	105.40
94	106.00	+0.10	106.10	+0.20	+0.90	106.10
95(P.T.)	107.00	+0.00	107.00	+0.10	(+1.00)	107.00
96	108.00			+2.00	=A	

bers, etc., on the sheet of profile paper. (2) Determine the grade angle, that is, the algebraic sum of the two rates of grade. (3) Determine the length of the vertical curve by dividing the grade angle by the assigned or adopted change of grade per station (notice the analogy to simple circular curves). (4) Calculate the apex correction. (5) Determine the corrections at the several station or fractional stations (as assigned), and tabulate the stations and elevations. (6) Plot the vertical curve from the data so determined, as in the example. (7) Also compute and plot the same curve by the method of chord gradients.

PROBLEM C6. ESTABLISHING A GRADE LINE.

(a) *Equipment.*—Leveling instrument, leveling rod, flag pole, 100-foot steel tape, stakes, axe.

(b) *Problem.*—Establish an assigned grade line, (1) by measured distances and calculate rod readings, and (2) by "shooting in" the same line, for comparison.

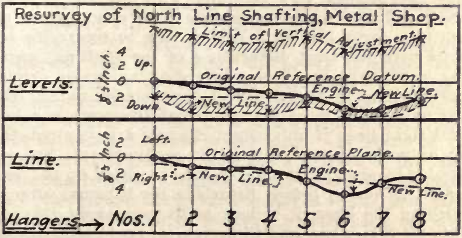
(c) *Methods.*—(1) Stake off the distance between ruling points, and drive stakes to the required grade, or if desirable, parallel to it, by dividing up the fall in proportion to the distance. (2) Set the level over one ruling point and determine the height from the point to the line of collimation by means of the leveling rod; set the flag pole behind the other ruling point and establish a target, consisting of a rubber band holding a strip of paper wrapped about the pole at a height equal to the rod reading; having thus constructed a line parallel to the desired grade line, direct the telescope on the fore sight target, and with the same rod reading, "shoot in" the same stakes. Make careful record of data and comparative results.

PROBLEM C7. SURVEY OF LINE SHAFTING.

(a) *Equipment.*—Engineers' transit with attached bubble, leveling rod (or instead of these engineers' instruments, a 16-foot metal-bound straight-edge with an adjustable bubble of say 20-foot radius, a long braided fishing line, and 3 long metal suspenders exactly alike (as shown in the form), to suspend straight-edge from line of shafting), 2 good plumb bobs, 50-foot etched steel tape, copper tacks, hatchet.

(b) *Problem.*—Make a survey of a line of shafting in a machine shop, and establish a true alinement for it, both vertically and transversely.

(c) *Methods.*—(Assuming that the transit and rod are not available), (1) Plan the survey carefully, and if possible find some well defined base line close to the line shafting, to which to refer its position transversely. (2) Stretch the braided string from end to end of the shaft line (or from end to end of the room in case the shafting passes through the wall), and fix it taut on or parallel to the adopted base of reference; upon finally fixing the location of the string, at least three points, one at either end and one near the middle of its length, should be carefully plumbed down and marked temporarily with copper tacks in the wooden floor of the shop for further reference; there should be as little draft as possible during this and the following steps. (3) Plumb down from the line shaft at each hanger and carefully measure the horizontal right angled distance to the reference string, noting the nearest 1-16-inch; the hangers should be numbered and the distances between them measured and recorded; the plumb bob should be suspended from corresponding points at all the hangers; and the bob should always hang from the same side of the shafting; likewise, the shafting should be calibrated, and record made of any changes of diameter found. (4) Determine the radius of curvature of the bubble on the straight-edge, (the radius should be at least 20 feet); test the parallelism of the edge of the straight-edge and the bubble line after the manner used with the carpenters' level, that is, by reversion, and



adjust the bubble if found in error; or if there is no way to adjust the bubble, find its mean position. (5) Prove the equality of the suspenders in a similar manner, by hanging them from the shafting and testing them with the adjusted straight-edge. (6) Having verified the special instruments, determine the relative elevations of the shafting at the successive hangers, noting the nearest 1-16-inch of elevation; the differences can be accurately measured by means of a wedge scale; for greater precision, the level may be reversed end for end each time, and the mean taken; reduce the level notes by summing up the differences algebraically so as to secure elevations relative to the first or any other hanger as a reference or bench, or above or below any datum plane desired; it is a good plan to adopt a marked spot on a machine or engine foundation as zero datum. (7) Plot the data on profile paper, so as to secure an exaggerated vertical and lateral profile; now inspect the several hangers and note the margin of adjustment available in the screws, making record of same on the profile; if the line of shafting passes through into another room of the shop, carry a line through a door or other opening on the prolongation of the reference line, using great care with the parallels, if any be required; collect complete data relative to the alinement through the length of the entire shafting, as described above for the first stretch of it; also plot any definite lines such as jack shafting lines or axes of long or important machines which may now or in the future bear a relation to the shafting now under survey. (8) Study the profiles very carefully with the aid of a fine thread; and after due consideration of all ruling points and conditions, lay a line on it with a view to secure the best results with the least disturbance of the shafting; abrupt turns or elbows are likely where shafting passes through small openings in partition walls, and sudden swings often occur near heavy machines; the ideal alinement is a horizontal right line; if only slight changes are required, they may be made at once, but if the readjustments are considerable in amount, it may be wise to check up the main lines of the survey before disturbing the hangers; after establishing the lines, it is best to fix permanent reference points for future use, and these points should be characteristic (such as one copper tack surrounded by three others), to avoid mistakes of identification; a line of tacks, one beneath the edge of each hanger, located say in a vertical plane tangent to the same side of the shafting throughout. establishes the element of the cylinder, which is more convenient to use than the axis of the shaft line. (9) Carefully preserve the record of the survey and changes of the hangers, and in due time make a resurvey to discover loose and shifting hangers, especially near belts under heavy stress.

(Should the regular engineers' instruments be employed, the general method would be unchanged; the difference would consist in the greater facility of securing the data, in passing through difficult places from one room of the shop to another, in reestablishing the alinement, and in detecting changes subsequently. As a rule, the resurvey should be made when the shafting is idle, and if a transit or level is employed, it should, when possible, be set up on a masonry foundation of an engine or machine to avoid disturbance from the shaky wooden floors due to the vibration of machinery elsewhere in the shop, or to the observer moving about the tripod legs.) Keep the record in tabular form and make profile in the manner indicated in the accompanying diagram.

PROBLEM C8. CONTOUR LEVELING.

(a) *Equipment.*—Engineers' leveling instrument, leveling rod, 100-foot steel tape, stakes, axe.

(b) *Problem.*—Make a rapid contour survey of an assigned tract of ground with the level and chain.

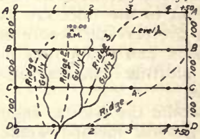
(c) *Methods.*—(1) Examine the tract and plan the system of reference lines for locating the points at which levels are to be taken; if the ground is comparatively regular, a simple subdivision into squares of 100 feet may suffice; but if much broken, special lines along gullies and ridges should be included in the survey plan. (2) Stake off the tract according to the plan, and make a record of the same. (3) Starting from an assigned bench, determine the elevations of the ground at the various stakes and at such other points as may be required to give a correct basis for accurate contouring. (4) Plot the data, and interpolate contours at a specified interval, employing both numerical calcula-

J. Doe, Leveler.
R. Roe, Rodman.

LEVELS FOR CONTOURS ON PROPOSED PARK SITE. Phila rod.

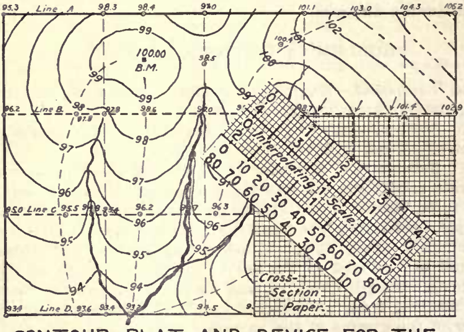
S.	+	⊼	−	R.	E.	
B.M.	6.67	106.67			100.00	Oct. 26, 1899. Clear, warm. B.&B. Wye Level. Boulder, Sta. A 1+45, 46'R., on knoll.
A 0				11.4	95.3	Ground. (All levels from single setting.)
A 1				8.4	98.3	"
A 1+45				8.3	98.4	"
A 1+45 (46'R.)				6.9	99.8	"
A 2				9.7	97.0	"
A 3				5.6	101.1	"
A 3+50 Ridge 3				3.7	103.0	"
A 4				2.4	104.3	"
A 4+50				0.5	106.2	"
B 0				10.5	96.2	"
B 0+75 Ridge 1.				8.9	97.8	"
B 1 Gully 1.				8.9	97.8	"
B 1+40 Ridge 2.				8.1	98.6	"
B 2 Gully 2.				9.7	97.0	"
B 2+40 Ridge 3.				8.2	98.5	"
B 2+70 Gully 3.				8.8	97.9	"
B 3				8.0	98.7	"
B 4				5.3	101.4	"
B 4+50				3.8	102.9	"
C 0				11.7	95.0	"
C 0+60 Ridge 1.				11.2	95.5	"
C 0+85 Gully 1.				11.9	94.8	"
C 1				11.3	95.4	"

(Continued on following pages.)



Dotted lines show ridges

Set stakes only at sta. 0 and 4+50 on each line for future reference. Used chaining pins for intermediate points



CONTOUR PLAT AND DEVICE FOR THE RAPID INTERPOLATION OF CONTOURS.

tions and geometrical methods. (5) Finish the plat, as required.

PROBLEM C9. USE OF CONTOUR MAP.

(a) *Equipment.*—Contour map, drafting instruments, etc.

(b) *Problem.*—From the given contour map: (1) construct profiles on the assigned lines; (2) project a line of specified grade through assigned points on the contour map; make profile, lay grade line and estimate earthwork quantities approximately; (3) calculate the earthwork quantities from the map for given grade planes and limitations of area. (The third step may, perhaps, best be taken with a different map from the first two.)

(c) *Methods.*—(1) Use profile paper for the profiles. (2) To project the line on the map, set the dividers at the horizontal distance in which the specified gradient will surmount the vertical interval between successive contour planes; then beginning at a specified point, locate points on the successive contour lines up or down on the given gradient, as required; sketch in the route roughly, and project a series of connected curved and tangent lines approximating to it; construct a profile along the new line; lay the required grade line on the profile, and estimate approximate earthwork quantities for specified dimensions and slopes of roadbed. (3) By means of end area method calculate the earthwork quantities required to establish the specified grade planes on the designated contoured area.

PROBLEM C10. TEST OF DELICACY OF BUBBLE VIAL.

(a) *Equipment.*—Engineers' leveling instrument, leveling rod, tape, level tester.

(b) *Problem.*—Determine the radius of curvature of the assigned bubble vial. (1) by means of the optical test, and (2) by the level tester.

(c) *Methods.*—(1) Measure off a base line say 100 feet long, set level at one end and hold rod on a peg driven at the other end; note the target movement corresponding to a given bubble movement, both in the same linear unit; calculate the radius by the method shown at (h), Fig. 18. (2) Set the level tester on a solid base and place the instrument on it, as indicated at (i), Fig. 18; by means of the micrometer head and known relations of the level tester, determine the angular equivalent in seconds for one division and also one inch movement of the bubble, from which calculate the radius of curvature of the vial in feet. Follow the form.

PROBLEM C11. COMPARISON OF LEVEL TELESCOPES.

(a) *Equipment.*—Five (or other specified number) engineers' levels (both wye and dumpy), leveling rod, metallic tape.

(b) *Problem.*—Make a critical examination and comparison of the telescopes of the assigned instruments.

(c) *Methods.*—Carefully read the discussion of the telescope in the text. Then compare the telescopes with reference to: (1) magnifying power; (2) chromatic aberration; (3) spherical aberration; (4) definition; (5) illumination; (6) flatness of fields; (7) angular width of field; (8) effective aperture of objective. Make tabulated record of comparisons, giving in separate columns; (a) locker number; (b) kind of level; (c) name of maker; (d) magnifying power; and so on for the other points examined.

PROBLEM C12. TESTS OF THE WYE LEVEL.

(a) *Equipment.*—Wye level, leveling rod, tape.

(b) *Problem.*—Test the essential relations and adjustments of the wye level.

(c) *Methods.*—Carefully note the construction of the assigned level and the positions of the elementary lines. Then following the methods outlined in the text, test the following adjustments (but do not disturb the adjusting screws): (1) The bubble, both as to the azimuth and altitude movements; find the position of the bubble when parallel to the element of the rings. (2) The line of collimation; its deviation from the axis in 400 feet. (3) The wyes; finding the position of the bubble when the vertical axis is vertical. Keep a neat and systematic tabulated record of observed numerical data, with explanation of the several adjustments.

J. Doe } Observers.
R. Roe }

DELICACY OF BUBBLE VIAL, B. & B. WYE LEVEL.

1st Method, With Level Tester.

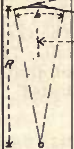
No.	Microm. Reading	Level Scale* A End.	Level Scale* B End.	Differences. A End.	Differences. B End.	Length Bubble
1	7	9.8	51.8			
2	17	14.2	47.2	4.4	4.6	61.4
3	27	18.5	42.7	4.3	4.5	61.2
4	37	23.0	38.5	4.5	4.2	61.5
5	47	27.0	34.4	4.0	4.1	61.4
6	57	31.5	30.0	4.5	4.4	61.5
7	67	35.8	25.7	4.3	4.3	61.5
8	77	40.2	21.0	4.4	4.7	61.2
8	77	40.0	21.2			61.2
7	67	35.5	25.5	4.5	4.3	61.0
6	57	31.1	30.2	4.4	4.7	61.3
5	47	26.7	34.6	4.4	4.4	61.3
4	37	22.3	38.8	4.4	4.2	61.1
3	27	18.2	42.8	4.1	4.0	61.0
2	17	14.0	47.1	4.2	4.3	61.1
1	7	9.5	51.5	4.5	4.4	61.0
				60.9	61.1	

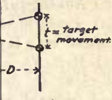
(* Level scale graduated to 20ths inch) Mean = $\frac{4.36}{20\text{ths}}$ inch

Ten divisions of micrometer screw correspond to 19.2 seconds of arc, so that one division of scale = $\frac{19.2}{4.36}$ = 4.4 seconds, and one division on bubble tube (tenths of an inch) = 8.8 seconds, or 88 seconds per inch.

Radius in feet = $\frac{17182}{\text{Secs. per inch}}$ = $\frac{17182}{88}$ = 195 feet.

2nd Method, With Telescope.





$R : D :: b : t$

$R = \frac{b}{t} D$

$R = \frac{bD}{t}$

$= \frac{.083 \times 100}{0.042}$

$= 198$ ft.

Target Movement (t) at 100 ft. (D) for Bubble Movement (b) of 1 Inch (0.083 ft.)

No.	Rods (ft.)		b (ft.)
1	4.631	4.588	0.043
2	4.586	4.546	0.040
3	4.547	4.591	0.044
4	4.593	4.634	0.041
	Mean. =		0.042

PROBLEM C13. ADJUSTMENT OF THE WYE LEVEL.

(a) *Equipment.*—Wye level (reserved expressly for adjustment), leveling rod, tape, adjusting pin.

(b) *Problem.*—Make the full series of adjustments of the wye level.

(c) *Methods.*—Follow the methods detailed in the text according to the following program: (1) Adjust the bubble line (a) into the same plane with the bottom element of the rings, and (b) parallel to that element. (2) Adjust the line of collimation to coincide with the axis of the rings, first on a long distance; and then, to test the object glass slide, try it for a short distance; if necessary, shift the reticule in rotation to make the horizontal hair horizontal, and also center the eyepiece. (3) Adjust the bubble line perpendicular to the vertical axis by means of the wye nuts. (4) Test the rings of the wye level by the two-peg test; if the level has a reversion bubble, first test the parallelism of the top and bottom tangent lines, and then test the rings. Keep a clear and systematic record. In each case, state (a) the desired relation, (b) the test, and (c) the adjustment.

PROBLEM C14. SKETCHING THE WYE LEVEL.

(a) *Equipment.*—Wye level.

(b) *Problem.*—Make a first-class freehand sketch of the assigned wye level.

(c) *Methods.*—The sketch should be correct in proportion and clear in detail. The essential parts should be designated in neat and draftsmanlike form, and the elementary lines clearly indicated.

PROBLEM C15. TESTS OF THE DUMPY LEVEL.

(a) *Equipment.*—Dumpy level, leveling rod, tape.

(b) *Problem.*—Test the essential relations and adjustments of the dumpy level.

(c) *Methods.*—Carefully note the construction of the assigned level and the position of the elementary lines. Then, following the methods outlined in the text, test the following adjustments: (1) the bubble line, whether perpendicular to the vertical axis; and if not, what is the angular inclination of the vertical axis when the bubble is in the middle? (3) The line of collimation, whether parallel to the bubble line. Record the errors and observations systematically.

PROBLEM C16. ADJUSTMENT OF THE DUMPY LEVEL.

(a) *Equipment.*—Dumpy level (reserved expressly for adjustment), leveling rod, tape, pegs, axe, adjusting pin.

(b) *Problem.*—Make the essential adjustments of the assigned dumpy level.

(c) *Methods.*—(1) Adjust the bubble line perpendicular to the vertical axis. (2) Adjust the line of collimation parallel to the bubble line by the two-peg method. In describing the adjustments, the record should state (a) the desired relation, (b) the test, and (c) the adjustment.

PROBLEM C17. SKETCHING THE DUMPY LEVEL.

(See Problem C14.)

PROBLEM C18. STRETCHING CROSS-HAIRS.

(a) *Equipment.*—Engineers' level or transit (or cross-hair reticule), pocket cross-hair outfit, reading glass.

(b) *Problem.*—Renew the cross-hairs in a level or transit instrument by a method applicable to field use.

(c) *Methods.*—(If instrument is provided, follow the complete program outlined below; otherwise, merely stretch the lines on the reticule and test same.) (1) Remove the eyepiece, carefully preserving the screws from loss. (2) Remove one pair of the capstan headed reticule screws; turn the ring edgewise and insert a sharpened stick in the exposed screw hole, take out the other two screws and remove reticule from telescope tube. (3) Clean the cross-hair graduations, and support the reticule on a sharpened stick, or (if a transit) place it on the object glass with a piece of paper interposed to protect the lens. (4) Select from the capsule (see (d), Fig. 17) two spider lines 2 inches or more long, and fasten a stick to either end of each hair by means of glue from the adhesive paper. (5) Put the hairs in place, (with the bits of wood hanging loose), shifting them as desired with a pin point or knife blade. (6) Apply a bit of the moistened adhesive paper to the reticule over each hair, and after a few minutes cut or break the sticks loose. (7) Test the hairs by blowing on them full force. (8) If they stand this test, replace the reticule, and adjust the instrument. Make a record of the process.

PROBLEM C19. ERROR OF SETTING A LEVEL TARGET.

(a) *Equipment.*—Engineers' leveling instrument, leveling rod (perferably a New York or Boston rod), tape, pegs.

(b) *Problem.*—Determine the probable error of setting the level target at distances of 100 and 300 feet (or such other distances as may be assigned.

(c) *Methods.*—(1) Determine the magnifying power of the telescope. (2) Determine the radius of curvature of the level vial by the field method. (3) Determine the space on the rod covered by the diameter of the hair. (4) Drive a peg at 100 feet from the level, level up, and secure ten satisfactory consecutive rod readings with rod held truly plumb on the peg; shift the target several inches between readings, and reset without bias; reject no readings; watch the bubble closely, but work briskly. (4) Repeat the series at 300 feet. (5) Determine for each distance the mean rod, the probable error of a single reading, and of the mean, as indicated in the form.

ERROR OF SETTING

Distance 100 Feet.				Distance 300 Feet.			
No.	Reading Ft.	d Ft	d²	No.	Reading Ft.	d	d²
1	3.169	0.000	0.000000	1	4.843	0.006	0.000036
2	3.170	.001	.000001	2	4.835	.002	.000004
3	3.170	.001	.000001	3	4.836	.001	.000001
4	3.170	.001	.000001	4	4.837	.000	.000000
5	3.169	.000	.000000	5	4.834	.003	.000009
6	3.168	.001	.000001	6	4.834	.003	.000009
7	3.171	.002	.000004	7	4.837	.000	.000000
8	3.168	.001	.000001	8	4.839	.002	.000004
9	3.169	.000	.000000	9	4.838	.001	.000001
10	3.168	.001	.000001	10	4.835	.002	.000004
n	3.169	0.0008	.000010	n	4.837	0.002	0.000068
	Mean.	Mean = f	Sum = Σ		Mean	Mean = f	Sum = Σ

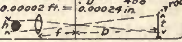
Distance 100 Feet.	Distance 300 Feet.
Prob. Error Single Obs. $E_1 = 0.67\sqrt{\frac{\Sigma d^2}{n-1}} = 0.00070$	$E_1 = 0.0016$
Approx. Prob. Error Single Obs $= 0.85f = 0.00068$	E_1 (approx) $= 0.0017$
Prob. Error of Mean $E_m = \frac{E_1}{\sqrt{n}} = 0.00022$	$E_m = 0.0005$

Leveler, R. Roe. Young Level.
Rodman, J. Doe. Boston Road, Lr. 12.
LEVEL TARGET.
Nov. 1, '38, (2 hours) Cloudy, cool, breezy.
Distances with 50-Ft. Metallic Tape.

Set instrument in sheltered place, measured off 100' and drove peg. Same at 300'.
Placed pair of foot screws on general line of pegs and leveled up, leaving screws just snug.
Focused eyepiece on crosshairs very carefully, keeping eye in normal condition.
Set target ten times at each distance, carefully verifying the position of the bubble each time before approving the sight.
Determined magnifying power of telescope by comparing 0.1 ft. on rod natural size with one eye and magnified by telescope with the other eye. Found Mag. Power to be 28 diameters.
Found radius of bubble curvature

$$R = \frac{b}{t}D = \frac{0.07}{0.048} \times 100 = 145.'8$$

Diam. hor. hair, $h = \frac{tf}{D} = \frac{0.01 \times 0.8}{400}$
$= 0.00002$ ft. $= 0.00024$ in.



PROBLEM C20. COMPARISON OF DIFFERENT MAKES AND TYPES OF ENGINEERS' LEVELS.

(a) *Equipment.*—Department equipment, catalogs of representative engineering instrument makers.

(b) *Problem.*—Make a critical comparison of the several types and makes of engineers' levels.

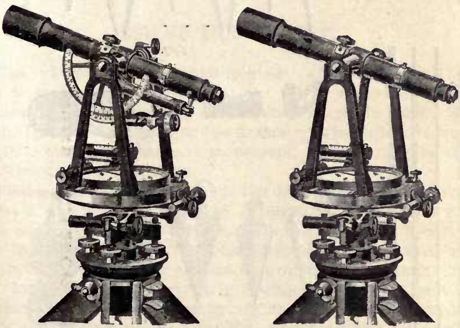
(c) *Methods.*—Examine the department equipment and study the several catalogs carefully, noting the usual and special features, prices, etc., and prepare a systematic summary or digest of the same. Prepare brief specifications for a leveling instrument, and also suggest the preferred make.

CHAPTER V.

THE TRANSIT.

Description.—The engineers' transit consists of an alidade, carrying the line of sight, attached to an inner vertical spindle (or upper motion) which turns in an outer annular spindle (or lower motion). The latter carries the horizontal graduated circle or limb, and is supported by the tripod head. The alidade includes the telescope, magnetic needle with its graduated circle, and the vernier; it may be revolved while the graduated limb remains stationary. The horizontal limb is graduated to degrees and half degrees and sometimes to twenty minutes, and is numbered preferably from zero to 360° in both directions.

The complete transit differs from the plain transit, Fig. 20, in having a vertical arc and level bubble attached to the telescope.



COMPLETE TRANSIT. PLAIN TRANSIT.

Fig. 20.

In Fig. 21 are shown: (a) the English theodolite; (b) the shifting plates and foot screws of a transit; (c) the Saegmuller solar attachment to the transit; (d) the grad-

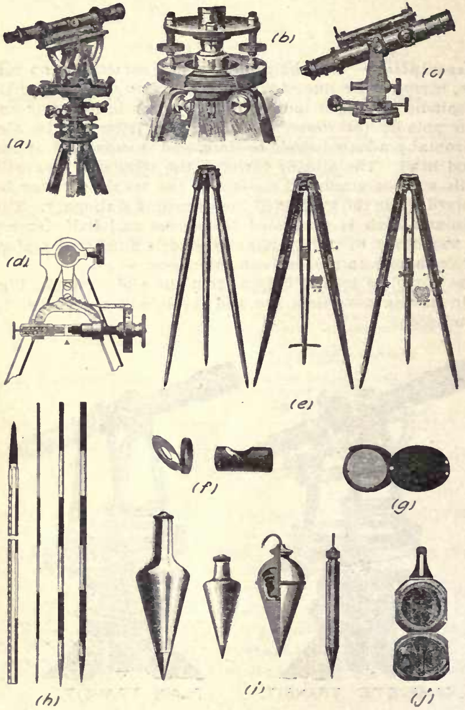


Fig. 21.

ienter; (e) tripods; (f) reflectors; (g) reading glass; (h) flag poles; (i) plumb bobs; (j) the Brunton pocket transit.

The Vernier.—The vernier is an auxiliary scale used to read fractional parts of the main graduated scale or limb. The *least count* of a direct vernier is found by dividing the value of one division of the limb by the number of divisions on the vernier. With a limb graduated to half degrees and a direct vernier reading to single minutes, 30 divisions on the vernier cover 29 divisions on the limb.

In *reading* a direct vernier observe the following rule: Read from the zero of the limb to the zero of the vernier, then along on the vernier until coincident lines are found. Add the reading of the vernier to the reading of the limb.

In *setting* the vernier to a given reading, as for example a zero reading for measuring an angle, the tangent movement should be given a quick short motion to secure the last refinement, since a slow movement is not noticed by the eye. Notice adjacent and end graduations.

In Fig. 23, (c) is a vernier reading to single minutes, (d) to half minutes (30″), and (e) to thirds of minutes (20″). The slant in the numerals on the limb corresponds with that on the vernier.

USE OF THE TRANSIT.

Use.—The complete transit is used: (1) to prolong lines; (2) to measure horizontal angles; (3) to measure vertical angles; (4) to run levels; (5) to establish grade lines. The plain transit is confined to the first two uses, unless it has a vertical clamp and tangent movement, when it may be used to "shoot in" grade lines.

Prolongation of Lines.—If the instrument is in adjustment a line can be prolonged by sighting at the rear station and reversing the telescope in altitude. It is, however, not safe to depend on the adjustments of the transit, and important lines should always be prolonged by the method of "double sights," as given in Problem D2. Lines may be prolonged with the plates by sighting at the rear station with the A vernier reading 180°, reversing the alidade in azimuth and locating stations ahead with the A vernier reading zero. A third method employs two points ahead of the instrument.

Measurement of Horizontal Angles.—Horizontal angles are measured as described in Problem D1. If greater accuracy is required, angles may be measured by series or by repetition.

By Series.—In measuring an angle by series all the angles around the point are read to the right, both verniers being read to eliminate eccentricity. The instrument is then reversed in altitude and azimuth and all the angles around the point are read to the left. The readings are checked by sighting back on the first point in each case. These observations constitute one "set." The vernier is shifted between sets 360° divided by the number of sets. The arithmetrical mean of the observed values is taken as the true value.

By Repetition.—Angles are measured by repetition as described in Problem D10. This method is especially suited to the accurate measurement of angles with an ordinary transit and is to be preferred to the series method which is a favorite where precise instruments are used. In the repetition method all the instrumental errors are eliminated and the error of reading is very much reduced. It is doubtful if it is ever consistent to make more than 5 or 6 repetitions.

Azimuth.—The azimuth of a line is the horizontal angle which it makes with a line of reference through one of its ends, the angles being measured to the right from 0° to 360°, as in (f) Fig. 23. It is usual to assume that the true meridian is the line of reference, the north point being taken as zero in common surveying.

Deflection.—The deflection of a line is the angle that it makes with the preceding line produced, and is called deflection right or left depending upon whether the angle is on the right or left side of the line produced, as in (h), Fig. 23.

Vertical Angles.—Vertical angles are referred to the horizon determined by the plane of the level under the telescope, and are angles of depression or elevation relative to that plane. In measuring vertical angles the instrument should be leveled by means of the level under the telescope and correction should be made for index error of the vernier. With a transit having a complete vertical circle, the true vertical angle may be obtained by measuring the angle with the telescope normal and reversed and taking the mean.

Traversing.—A traverse is a series of lines whose lengths and relative directions are known. Traverses are used in determining areas, locating highways, railroads, etc.

Azimuth Traverse.—In an azimuth traverse the azimuths of the lines are determined, usually passing around the field to the right. In *orienting* the transit at any station the A vernier is set to read the azimuth of the preceding course, the telescope is reversed, directed towards the preceding station and the lower motion clamped; the telescope is then reversed in altitude. The reading of the A vernier with telescope normal will then give the azimuth of any line sighted on. If there is any error in collimation the transit may be oriented by sighting back with the A vernier reading the back azimuth of the preceding course. In a *closed traverse* the last front azimuth should agree with the first back azimuth. The azimuth traverse is especially adapted to stadia and railroad work. Azimuths can be easily changed to bearings, if desired.

Deflection Traverse.—In a deflection traverse the deflection of each line is determined, usually passing around the field to the right. To avoid discrepancies due to error in collimation, the transit may be oriented by sighting at the preceding station with the A vernier set at 180°, the telescope being in its normal position, and the lower motion clamped. The reading of the A vernier will then give the deflection of any line sighted on.

Compass Bearings.—Compass bearings should always be read on an extended traverse as a check against such errors as using the wrong motion or an erroneous reading of the vernier. To guard against errors due to local attraction, back and front bearings should always be read, and the angle thus determined compared with the transit angle.

Leveling with the Transit.—The transit with an attached level is the complete equivalent for the engineers' level. The instrument is leveled up with the plate levels first, after which the position of the attached bubble is controlled by means of the vertical tangent movement.

Grade Lines.—Grade lines may be established with the transit either by means of known distances and calculated rod readings, or by "shooting in" a parallel line by means of the inclined telescope, as described under the use of the engineers' level. For the latter purpose the transit is rather more convenient than the level.

Setting up the Transit.—To set the transit over a point spread the legs so that they will make an angle of about 30°, place them symmetrically about the point with two legs down hill. Bring one plate level parallel to two of the legs, force these legs firmly into the ground and bring the plumb bob over the point and the plates approximately level with the third leg, changing the position of the plumb bob with a radial motion and leveling the plates with a circular motion of the leg. Finish the centering with the shifting plates. In leveling up the bubbles move with the left thumb. Use care to bring the foot screws to a proper bearing.

Parallax.—Before beginning the observations the eyepiece should be carefully focused on the cross-hairs so as to prevent parallax.

Back Sight With Transit.—*Always check the back sight before moving the transit* to see that the instrument has not been disturbed or that a wrong motion has not been used.

Instrumental Errors.—The transit should be kept in as perfect adjustment as possible, and should be used habitually as though it were out of adjustment, that is, so that the instrumental errors will balance. No opportunity should be lost to test adjustments.

ADJUSTMENTS OF THE TRANSIT.

Elementary Lines.—Fig. 22 shows the elementary lines of the transit, viz., (1) line of collimation; (2) horizontal axis; (3) vertical axis; (4) plate level lines; (5) attached level line. These lines should have the following relations: (a) the plate levels should be perpendicular to the vertical axis; (b) the line of collimation should be perpendicular to the horizontal axis; (c) the horizontal axis should be perpendicular to the vertical axis; (d) the attached level line should be parallel to the line of collimation. The following additional relations should exist: (e) the vertical axes of the upper and lower motions should be coincident; (f) the optical center of the objective should be projected in the line of collimation; (g) the center of the graduated circle should be the center of rotation, i. e., there should be no eccentricity.

Plate Levels.—*To make the plate levels perpendicular to the vertical axis.—Make the vertical axis vertical and adjust the bubbles to the middle of their race.* The vertical axis is made vertical by leveling up, reversing in azimuth, and if the bubbles move, bring them half way back with the foot screws. The adjustment is the same as for the compass, and the reasons are shown in (a), Fig. 13.

After adjusting the plate levels with reference to say the upper motion, test them with the lower motion to prove the coincidence of the vertical axes.

Line of Collimation.—*To make the line of collimation perpendicular to the horizontal axis.—Construct a straight line*

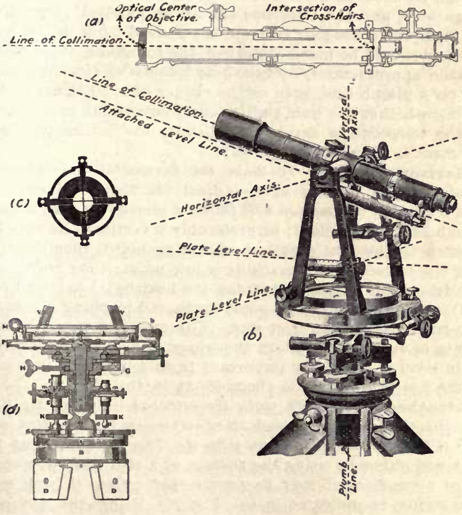


Fig. 22.

and adjust the vertical hair so that the instrument will reverse in altitude on it. The straight line may be established either by prolongation beyond a point in front, or preferably by the methods of double sighting, described in Problem D2. One-fourth the apparent error is corrected for the reasons indicated in (a), Fig. 23. In deciding which way to move the hair, notice that the optical center is the fulcrum. The transit should be collimated first for equal back and fore sights, say 100 feet or so, and then checked for a distant point in one direction and perhaps 50 feet in the other, so as to test the motion of the optical center of the objective. The points should all be as definite as possible. Chaining pins may be used, or V-marks may be made on the side of a stake driven securely. Each altitude reversal should be checked back and forth to make sure of the prolongations, and the telescope should be handled very carefully. If the cross-hair reticule is removed from the instrument or should be much disturbed, the foregoing adjustment is made approximately and the hair is made vertical by sighting on a plumb line, such as the corner of a building, or by noting whether the hair continuously covers the same point as the telescope is moved in altitude; the collimation adjustment is then made precisely.

Horizontal Axis.—*To make the horizontal axis perpendicular to the vertical axis.—Adjust the horizontal axis so that the line of collimation will follow a plumb line.* An actual plumb line may be used; or preferably a vertical line may be constructed by first sighting on a high point, then depressing the telescope and marking a low point; then reversing in altitude and azimuth (turning the horizontal axis end for end), sighting at the high point again and marking a second low point beside the first one. The mean of the two low points is vertically beneath the upper one. The transverse plate level is especially important in this process. One end of the horizontal axis is changed, as in (b), Fig. 23.

Attached Level.—*To make the attached level and the line of collimation parallel to each other.—Construct a level line and adjust the instrument to agree with it.* The level line may be obtained either by using the surface of a still body of water, as of a pond, or it may be constructed by equal back and fore sights, as indicated in (e), Fig. 16. Either the horizontal hair may be changed to bring the line of collimation

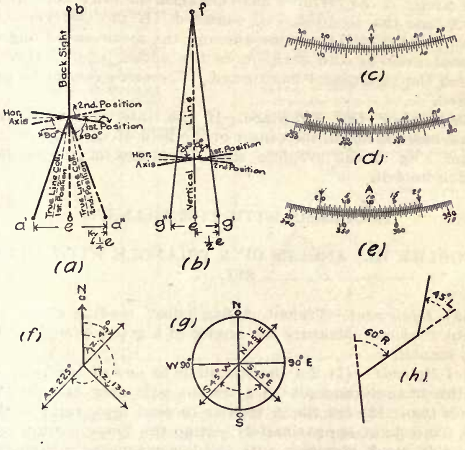


Fig. 23.

parallel to the bubble line, or vice versa. The method is the same as used for the dumpy level.

If the bubble vial is a reversion level, as shown at (b), Fig. 18, the adjustment is much simpler. However, the two-peg test should be applied *at least once* to the reversion level to prove the parallelism of the top and bottom tangent lines of the bubble vial.

Vertical Arc.—After the last preceding adjustment, the vernier of the vertical circle should be made to read zero when the bubble is at the center of the tube. Bring the bubble to the center and shift the vernier to read zero. If the vernier is fixed, an index correction may be applied to all vertical angles; or the bubble may be made to agree with the vernier and the horizontal hair then adjusted by the two-peg method.

Eccentricity.—Read the two verniers at intervals around the circle; if the verniers have changed the same amount in each case the circle is well centered. If the two verniers have not changed the same amount, the mean of the angles passed over by the verniers is the actual angle through which the instrument has turned. The error cannot be adjusted.

Centering the Eyepiece.—If the intersection of the cross-hairs is not in the center of the field of view, move the inner ring of the eyepiece slide by means of the screws which hold it.

PROBLEMS WITH THE TRANSIT.

PROBLEM D1. ANGLES OF A TRIANGLE WITH TRANSIT.

(a) *Equipment.*—Transit, 2 flag poles, reading-glass.

(b) *Problem.*—Measure the angles of a given triangle with the transit.

(c) *Methods.*—(1) Set the transit over one of the vertices of the triangle and plumb a transit pole over each of the other two. (2) Set the A vernier to read zero, sight at the left hand point approximately, clamp the lower motion and make an exact bisection with the lower tangent movement. (3) Unclamp the upper motion, sight at the right hand point approximately and make an exact bisection with the upper tangent movement. (4) Read the A vernier to the nearest single minute. This reading is the angle sought. (5) With the A vernier set to read zero repeat the measurement, sighting first at the right hand station and then at the left. The recorded value of the angle is to be the mean of these two determinations which must not differ by more than one minute. (6) Measure the other angles in like manner. The error of closure must not exceed one minute. Follow the prescribed form.

PROBLEM D2. PROLONGATION OF A LINE WITH TRANSIT.

(a) *Equipment.*—Transit, 2 flag poles, axe, 6 hubs, 6 flat stakes, tacks.

ANGLES OF TRIANGLE 5-6-8

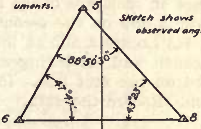
Station	Value of Angle 1st. Meas.	2d Meas.	Mean
5	88° 50′	88° 51′	88°50′30″
6	~~47 47~~	~~47 50~~	
″	47 47	47 47	47 47 00
8	43 23	43 23	43 23 00
			180°00′30″

(Difference between measurements not to exceed 1′)
(Error not to exceed 1′)

Observers { J. Doe. / R. Roe.

WITH ENGINEERS' TRANSIT

Nov. 15, 1899, (2 hours). Warm and quiet.
Used Heller & Brightly Transit, No. 10.
The 1st. Measurement was made by sighting on Sta. 8 with the lower motion, the plates clamped at zero; then sighting on Sta. 6 with the upper motion, and reading the plates.
The 2d Measurement was made by sighting on Sta. 6 and then on Sta. 8.
Used transit poles as targets, plumbing them very carefully over the monuments.



Sketch shows observed angles.

(b) *Problem.*—Prolong a 300-foot base line successively with the transit by the method of "double sights" about 1500 feet, and check on a hub previously established.

(c) *Methods.*—(1) Drive two hubs, A and B, about 1500 feet apart. (2) Set the transit over tack in hub A, sight at flag pole plumbed over tack in hub B, drive hub C about 300 feet from the transit and locate a tack in line very carefully. Remove the flag pole from hub B. (3) Set the transit over hub C, back sight on hub A and clamp the vertical axis. (4) Reverse the telescope, drive hub D at a distance of about 300 feet and mark line very carefully with a pencil. (5) Reverse the transit in azimuth, sight on hub A; reverse the telescope and locate a second point on hub D. Drive a tack midway between these two points. (6) Set the transit over the mean point on hub D, back sight on hub C, prolong 300 feet and set hub E by double sights. (7) Set over hub E, back sight on hub D, prolong 300 feet and set hub F, as before. (8) Finally prolong from hub F, with back sight on E, and establish mean tack at terminal hub B. Record the collimation errors at D, E, and the final error at B. Follow the form.

PROBLEM D3. INTERSECTION OF TWO LINES WITH TRANSIT.

(a) *Equipment.*—Transit, 2 flag poles, plumb line string, axe, 6 hubs, 6 flat stakes, tacks.

(b) *Problem.*—Determine the intersection of two lines with the transit.

(c) *Methods.*—(1) Set the transit over a hub and sight at a second point on line. (2) Set and tack a hub on line a short distance on each side of the intersection. (3) Set the transsit over a hub on the second line and sight at a point on line. (4) Locate a hub at the intersection by sighting with the transit and stretching a string between the two hubs located on the first line. (5) Measure the angle of intersection. Record the data.

PROBLEM D4. TRIANGULATION ACROSS A RIVER.

(a) *Equipment.*—Transit, 2 flag poles, 100-foot steel tape, axe, 4 hubs, 4 flat stakes, tacks.

(b) *Problem.*—Determine the distance across an imaginary river by triangulating with the transit and check by direct measurement.

(c) *Methods.*—(1) Set the transit over a hub on line on one bank, and set a hub on the opposite bank of an imaginary river about 800 feet wide by "double sigths". (2) Turn off 90° and lay off a base line very carefully with the steel tape. (3) Set the transit over the hub at the farther end of the base line and measure the angle between the lines joining it and the other points. (4) Compute the distance across the river. (5) Measure the distance across the river and compare with the computed distance. The difference should not be greater than 1:1000. Follow the prescribed form.

PROBLEM D5. PASSING AN OBSTACLE WITH TRANSIT.

(a) *Equipment.*—Transit, 100-foot steel tape, 2 flag poles, axe, hubs, flat stakes, tacks.

(b) *Problem.*—Prolong a line beyond an imaginary obstacle by three methods and check by direct measurement.

(c) *Methods.*—(1) Pass the obstacle to the right by means

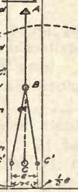
PROLONGATION OF A LINE

Transit at Sta.	Magnetic BackSight	Magnetic ForeSight	Transit Error, e Ft.	Set Stations
A		N30°00'E		F & B
B	S30°10'W	N30°10'E	0.20	C
C	S28°55'W	N28°55'E	0.24	D
D	S30°00'W	N30°00'E	0.18	E
E	S30°20'W	N30°20'E	0.30	F

Error in checking on F 0.01 ft
(Allowable Error 0.03 ft)

METHOD OF "DOUBLE SIGHTS"

Set the Transit over ⊙B and sight at △A; reverse in altitude and locate C'; reverse in azimuth and sight at △A; reverse in altitude and locate C''. The hub ⊙ C, set midway between C' and C'' will be on the prolongation of the line AB. The error of adjustment of the line of collimation is ½ e.



Observers {J. Doe. R. Roe.

WITH ENGINEERS' TRANSIT.

Nov. 16, 1899, (2 hours), Cool and Cloudy.
Used Keuffel & Esser Transit, Locker No. 4.
Set Transit over △A and sighted at a transit pole set at △F at a distance of 1500 ft., and set a hub at ⊙B in line with △A and △F and about 300 ft. from △A.
Set Transit over ⊙B and set hub ⊙C at a distance of about 300 ft. by "Double Sights." (see left hand page)
Set hubs ⊙D and ⊙E and checked △F, using the same method.

Note. It is very important to plunge telescope delicately, and to check back and forth on each sighting before approving point finally.

TRIANGULATION ACROSS A RIVER

Station	Distance Ft.	Angle	Value
B	150.00	D-B-C	90°00'
C		B-C-D	50°30'
BD	181.85		

Calculation of B D.

$BD = BC \times \text{Tan } 50°30'$
$\text{Log } BD = \text{Log } 150.0 + \text{Log. Tan } 50°30'$
$\text{Log } BD = 2.17609 + 10.08390$
$= 2.25999$
$BD = 181.96 \text{ ft.}$

$BD = BC \times \text{Tan. } 50°30'$
$= 150.00 \times 1.21310$
$BD = 181.96 \text{ ft.}$

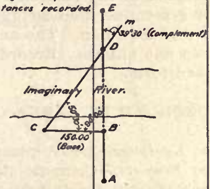
SUMMARY.

Computed result	= 181.96 ft.
Chained distance	= 181.85 "
Difference	= 0.11
l:d	= 1:1650
Permissible l:d	= 1:1000

Observers {J. Doe. R. Roe.

WITH ENGINEERS' TRANSIT.

Nov. 27, 1899. (2 hours), Cold and Clear.
Used Fauth Transit, Locker No 8; and Chaining Locker No 33.
With Transit over B set hub at D by the "Method of Double Sights," with A as a backsight.
Set hub at C, measuring BC with care, and measured ∠ BCD
Checked computed distance by chaining BD.
Length of Tape 99.98 ft. Observed distances recorded.



of the "equilateral triangle method" with sights of not less than 200 feet. (2) Pass the obstacle to the right by means of the "right angle off-set method" and check on the same hub as before. (3) Pass the obstacle to the left by means of the "deflection method", turning off an angle that will just pass the obstruction. Check the three methods by direct measurement. Follow the prescribed form.

PROBLEM D6. TRAVERSE OF FIELD WITH TRANSIT.

(a) *Equipment.*—Transit, 2 flag poles, 100-foot steel tape.

(b) *Problem.*—Determine the deflections of the sides of an assigned field with the transit, check angles by observing the magnetic bearings, and measure the lengths of the sides with a steel tape.

(c) *Methods.*—(1) Set the transit over one corner of the field, set the A vernier to read 180°, and sight at a flag pole plumbed over the point to the left with the telescope normal. Read and record the magnetic bearing. (2) Keep the telescope normal and sight at the next point to the right. The reading of the A vernier will be the deflection of the second line. (3) Read and record the magnetic bearing and compare the transit and magnetic deflections. (4) Repeat this process for the remaining corners of the polygon taken in succession to the right. Deflections will be based on duplicate readings agreeing within one minute. (5) Measure the sides to the nearest 0.01 foot with the tape. Compare the tape with the standard at the beginning and conclusion of the chaining. (6) From the observed deflections determine the bearings of the field assuming one side as a true meridian. The angular error of closure must not exceed one minute. Record and reduce data as in the prescribed form.

PROBLEM D7. AREA OF FIELD WITH TRANSIT.

(a) *Equipment.*—Five-place table of logarithms.

(b) *Problem.*—Compute the area of the assigned field by means of latitudes and departures.

(c) *Methods.*—(1) Prepare forms for calculation; transcribe data, and carefully verify copy. (2) Compute latitudes and departures by contracted multiplication, preserv-

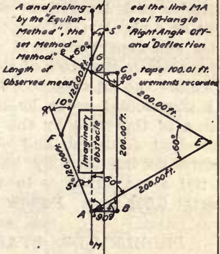
PASSING AN OBSTACLE

Station	Distance Ft.	Angle	Value	Error of Closure. Dist. Ft.	Line. Ft.
		"Equilateral Triangle Method."			
A	200.00	N-A-E	60°00′		
E	200.00	A-E-D	60°00′		
D	301.03	e-D-N	59°59′		
A-D	199.93			-0.07	0.09R
		"Right Angle Offset Method."			
A	20.00	N-A-B	90°06′		
B	200.00	A-B-C	90°00′		
C	20.00	B-C-D′	90°00′		
D′	300.66	C-B-N	90°01′		
A-D	200.08			+0.08	0.10L
		"Deflection Method"			
A	120.00	N-A-F	5°00′		
F	120.00	a-F-G	10°00′		
G	261.66	f-G-N	5°00′		
A-G	239.05			-0.03	0.03R

Observers {J. Doe. / R. Roe.

WITH ENGINEERS' TRANSIT.

Nov. 17, 1899, (2 hours), Warm and Cloudy.
Used Gurley Transit, Locker No. 5, and Chaining Locker No. 32.
With the Transit over M, set hubs at A and N, in the line MN. Set Transit at A and prolonged the line MA by the "Equilateral Triangle Method", the "Right Angle Offset Method" and "Deflection Method."
Length of tape 100.01 ft.
Observed measurements recorded.



TRAVERSE OF FIELD A-B-C-D-E- WITH ENGINEERS' TRANSIT, DEFLECTION METHOD

Station Inst.	Obj.	Distance Ft.	Deflection Angle	Magnetic Bearings	Check Angle	Calculated Bearings
A	E		86°31′R	N60°35′E		
	B	335.00		S32°45′E	86°40′R	S00°00′E
B	A		10°13′L	S33°05′E		
	C	464.96		S43°25′E	10°20′L	S10°13′E
C	B		124°53′R	S43°20′E		
	D	483.72		S61°35′W	124°55′R	N65°20′W
D	C		76°03′R	S61°36′W		
	E	616.53		N22°20′W	76°05′R	N10°43′E
E	D		82°46′R	N22°25′W		
	A	242.84		N60°10′E	82°35′R	S86°31′E
			370°13′		370°15′	
			10°13′		10°20′	
			360°00′	(Check)	359°55′	

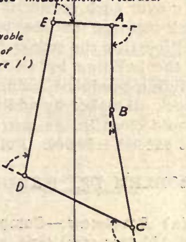
Calculation of Bearings

AB	S00°00′E	DE	N10°43′E	
B	10°13′L	E	82°46′R	
BC	S10°13′E	EA	S86°31′E	
C	124°53′R	A	86°31′R	Check
CD	N65°20′W			
D	76°03′R			
DE	N10°43′E			

Observers {J. Doe. / R. Roe.

Nov. 20, 1899, (2 hours), Warm and Misty.
Used Keuffel & Esser Transit, Locker No. 4.
Assumed that AB was a True Meridian.
Carefully checked each angle.
Bubble down on all sights.
Length of tape = 100.01 ft.
Reduced measurements recorded.

(Allowable Error of Closure 1′)



ing results to the nearest 0.01 foot. (3) Make the same calculations by logarithms as a check. (4) Determine the actual linear error of closure. (4) Determine the permissible error of closure (see chapter on errors of suveying). (6) If consistent, distribute the errors in proportion to the several latitudes and departures, respectively, repeating the additions as a check. (7) Copy the field notes and adjusted latitudes and departures, and verify transcript. (8) Calculate the meridian distances of the several stations and lines. (9) Calculate the latitude coordinates. (10) Calculate the partial trapezoidal areas by multiplying the meridian distances of the lines by the respective latitudes, preserving consistent accuracy, and observing algebraic signs. (11) Determine the area by taking the algebraic sum of the partial areas. Reduce to acres, preserving results to the nearest 0.001 acre. Follow the prescribed forms.

PROBLEM D8. STAKING OUT A BUILDING.

(a) *Equipment.*—Transit, 100-foot steel tape, 2 flag poles, axe, hubs, tacks, plan of building.

(b) *Problem.*—On an assigned plot of ground stake out the assigned building.

(c) *Methods.*—(1) Orient one side of the enclosing rectangle with reference to a true meridian or a street line. (2) Locate and check up the corners of the rectangle, by setting over each corner in turn, passing around to the right, back-sighting on the corner to the left, turning off 90° and locating the corner to the right. (3) Locate the corners of the building by setting stakes on the side lines of the building produced, using the rectangle as a base line. (4) Check all stakes by additional measurements. The rectangle should close to the nearest minute, the linear error should not exceed 1:50,000. Follow the prescribed form.

PROBLEM D9. HEIGHT OF TOWER WITH TRANSIT.

(a) *Equipment.*—Complete transit, 2 flag poles, leveling rod, 100-foot steel tape, axe, hubs, tacks.

(b) *Problem.*—Determine the height of an assigned tower with the transit and steel tape.

(c) *Methods.*—(1) Set the transit over a hub located a little

Nov. 21, '99 Data from pp. — Computer J. Doe — Transcript O.K.

TRANSIT TRAVERSE OF FIELD A-B-C-D-E — LATITUDES AND DEPARTURES.

Line	Adjusted Bearing	Reduced Distance Ft.	Computation of Latitudes: Multiplication	Logarithms	Computed Latitude Ft.	Lat. Cor. Ft.	Adjusted Latitude Ft.	Computation of Departures: Multiplication	Logarithms	Comput'd Departure Ft.	Dep. Cor. Ft.	Adjusted Departure Ft.
AB	S00°00'	335.05	....		S335.05	−.10	S334.95			000.0	.00	000.00
BC	S10°13'E	464.98	464.98 414.83 416482 37198 1860 47 19 457.696	2.66744 9.99306 2.66050 (457.61)	S457.61	−.10	S457.51	464.98 737 77 46498 32549 3255 139 33 82.474	2.66744 9.24888 1.91632 (82.47)	E 82.47	+.03	E82.50
CD	N65°25'W	483.72	463 72 206 14 193488 4837 2902 10 201.237	2.68459 9.61911 2.30370 (201.24)	N201.24	+.05	N201.29	483 72 639 09 435348 4353 145 29 439.875	2.68459 9.95873 2.64330 (439.87)	W439.87	−.18	W439.69
DE	N10°43'E	616.53	616.53 652 89 554877 49322 1233 308 37 605.777	2.78995 9.99236 2.78231 (605.78)	N605.78	+.15	N605.93	616.53 59581 61653 49322 3083 555 31 114.644	2.78995 9.26940 2.05935 (114.64)	E114.64	+.05	E114.69
EA	S88°31'E	242.84	242.84 67060 14570 170 15 14.755	2.38532 8.78361 1.16893 (14.76)	S14.76	+.00	S14.76	242.84 02839 218556 21856 1943 49 242.404	2.38532 9.99920 2.38452 (242.40)	E242.40	+.10	E242.50
	P =	2143.12			N807.02 S807.42 S 0.40	.40	N807.22 S807.22 0.00			E439.51 W439.87 W 0.36	.36	E439.68 W439.68 0.00

Error of Closure
Actual 0.55 ft.
Permissible 0.6 ft.
(See Diagrams)

Line	Ratio	Cor.
AB	2	.10
BC	2	.10
CD	1	.05
DE	3	.15
EA	0	.00
	8	.40

$\frac{.40}{8} = .05$

Line	Ratio	Cor.
AB	0	.00
BC	2	.03
CD	10	.18
DE	3	.05
EA	6	.10
	21	.36

$\frac{.36}{21} = .017$

STAKING OUT BUILDING WITH ENGINEERS' TRANSIT.

Surveyors, { John Doe, Richard Roe

Length of tape = 100'00", this tape being assumed as standard for the construction of this building.

Located hub A to fit the site, and then established a true meridian thro' A by observation on Polaris.

Then constructed a checked rectangle ABCD as follows:

Set Transit over hub A and set hub B and temporary hub D; set hub on line. Measured all distances twice.

Set Transit over hub B, sighted at A and set temporary hub C; set hub on line.

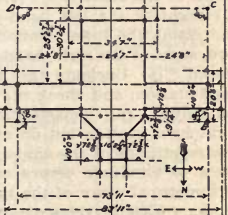
Set Transit over C, sighted at B and checked temporary hub D for angle and distance; error ½" for line, 1/32" for distance.

The remainder of the hubs were located with reference to the checked rectangle ABCD.

Nov 28, 1899. (3 hours), Cool and Clear.

Used Gurley Transit, Locker No. 8. & Locker No. 30.

Hubs are set on line, 5' from corners.



Plan of Observatory

further from the base than the height of the tower. (2) Level the instrument very carefully with the attached level and determine the index error of the vertical circle. (3) Bring the bubble of the attached level to the center and read a level rod held on the base of the tower. (4) Sight at the top of the tower, read the vertical angle, correct for index error and record. (5) Reverse the telescope and locate a second point at least as far from the first as the height of the tower, check by "double sights." (6) Set the transit over the second hub, sight at the top of the tower and read the vertical angle, as before. (7) Read the level rod on the base of the tower as before. Each angle and rod reading is to be based on duplicate readings. Follow the prescribed form.

PROBLEM D10. ANGLES OF TRIANGLE BY REPETITION.

(a) *Equipment.*—Transit, reading glass, 2 chaining pins, 2 tripods with plumb bobs (if necessary).

(b) *Problem.*—Measure the angles of a prescribed triangle with transit by repetition.

(c) *Methods.*—(1) Set the transit over one of the vertices of the triangle and set chaining pins in the tops of the monuments at the other two. (2) Set the A vernier to read zero, (3) Sight at the left hand station with the bubble down, and clamp the lower motion. (4) Unclamp the upper motion, sight at the right hand station, read both verniers and record. (5) Unclamp the lower motion, sight at the left hand station, and check the verniers to see that they have not moved. (6) Unclamp the upper motion and sight at the right hand station but do not read verniers. Repeat until five repetitions of the angle are secured, and read both verniers to eliminate errors of eccentricity. (7) Divide the arithmetrical mean of the two vernier readings by five and compare with the value obtained by single measurement. (8) Reverse the instrument in altitude, and set the A vernier to read zero. (9) Sight at the right hand station with the bubble up, and clamp the lower motion. (10) Unclamp the upper motion, sight at the left hand station, read both verniers and record. (11) Unclamp the lower motion, sight at the right hand station, and check the verniers to see that

HEIGHT OF TOWER

Station	Vertical Angle	D_1-D_2 Ft.	F.S. (Levels.)	
A	20°16′	150.00	4.50	$=h_1$
B	46°24′		3.82	$=h_2$

Calculation of Height.

(1) $D_1 = H_1 \operatorname{Cot.} M$

(2) $D_2 = H_2 \operatorname{Cot.} N$

(3) $H = H_1 + h_1 = H_2 + h_2$

(4) $H_1 = H_2 - (h_1 - h_2)$; substituting (4) in (1) and subtracting (2) from (1),

(5) $D_1 - D_2 = H_2(\operatorname{Cot} M - \operatorname{Cot} N) - (h_1 - h_2)\operatorname{Cot} M$

(6) $H_2 = \dfrac{D_1 - D_2 + (h_1 - h_2)\operatorname{Cot} M}{\operatorname{Cot} M - \operatorname{Cot} N}$

Substituting,

$$H_2 = \frac{150 + (4.50 - 3.82)\operatorname{Cot} 20°16'}{\operatorname{Cot} 20°16' - \operatorname{Cot} 46°24'}$$

$$= 86.47 \text{ ft.}$$

$$H = H_2 + h_2 = 86.47 + 3.82$$

$$= 90.29 \text{ ft.}$$

Observers { J. Doe. / R. Roe.

WITH ENGINEERS' TRANSIT.

Nov. 28, 1899, (2 hours). Warm and Cloudy.

Used Gurley Transit, Locker No. 5, and Chaining, Locker No. 35.

Set Transit over A and measured the vertical angle M, having first determined the index error of vertical circle.

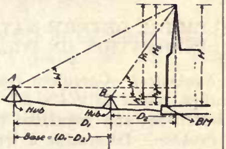
Read level rod on base of Tower. (h_1)

Set B in line with A and top of tower and measured D_1-D_2 as base line.

Set Transit over B and found N and h_2

Length of tape = 99.92 ft.

Reduced measurements recorded.



ANGLES OF TRIANGLE 5-6-8 BY REPETITION. Buff & Berger Transit No. 5.

Observers { John Doe. / Richard Roe.

Station	Bubble	Direction	Object	Vern. A.	Vern. B.	Mean	Difference	Angle	Mean Angle	Remarks
△6	Down	Right	△5	180°00′00″	0°00′00″	00′00″	Nov. 30, 1899, (2 hours), Cool and quiet.			
			△8	227 47 20	47 47 20	47 20	47°47′20″			Single
				58 56 20	238 56 20	56 20	238°56 20	47°47′16″		5 Reps.
	Up	Left	△8	0 00 00	180 00 00	00 00				
			△5	47 47 00	227 47 00	47 00	47°47′00″			Single
				238 56 40	58 56 40	56 40	238 56 40	47°47′20″	47°47′18″	5 Reps.
									Buff & Berger Transit No. 9.	
△8	D	R	△6	0°00′00″	180°00′00″	00′00″	Dec. 1, 1899, (3 hours) Warm and quiet.			
			△5	43 22 20	223 22 20	22 20	43°22′20″			Single
				216 52 00	36 52 00	52 00	216 52 00	43°22′24″		5 Reps.
	U	L	△5	180 00 00	0 00 00	00 00				
			△6	223 22 20	43 22 20	22 20	43°22′20″			Single
				36 51 40	216 51 40	51 40	216 51 40	43°22′20″	43°22′22″	5 Reps.
△5	D	R	△8	0°00′00″	180°00′00″	00′00″				
			△6	88 50 20	268 50 20	50 20	88°50′20″			Single
				84 12 40	264 12 40	12 40	444 12 40	88°50′32″		5 Reps.
	U	L	△6	180 00 00	0 00 00	00 00				
			△8	268 50 20	88 50 20	50 20	88°50′20″			Single.
				264 12 20	84 12 20	12 20	444 12 20	88 50 28	88°50′30″	5 Reps.
							(Error not to exceed 15″)		180°00′10″	

they have not moved. (12) Unclamp the upper motion and sight at the left hand station, but do not read the verniers. Repeat until five repetitions of the angle are secured, and read both verniers to eliminate errors of eccentricity. (13) Divide the mean of the two vernier readings by five and compare with the value obtained by single measurement (14) Take the mean of the two sets as the most probable value. (15) Measure the other angles in the same manner. The angular error of closure should not exceed 15″. Follow the prescribed form.

PROBLEM D11. DETERMINATION OF TRUE MERIDIAN BY OBSERVATION ON POLARIS AT ELONGATION.

(a) *Equipment.*—Complete transit, reading glass, hub, 2 flat stakes, plank 18″x 4″x 2″, 4 8d nails, axe, 2 lanterns, good watch set and regulated to keep railroad time.

(b) *Problem.*—Determine a true meridian by an observation on Polaris at elongation.

(c) *Methods.*—(1) Calculate the time of elongation of Polaris, and regulate and set a good reliable watch to keep railroad time (mean solar time for the 90th meridian.) (2) Set the transit over a hub about 40 minutes before the time of elongation. (3)Level the instrument very carefully, and set the vernier of the vertical circle to read the latitude of the place. (2) Focus the objective on a bright star; sight at Polaris which will be found by following the pointers of the Great Dipper at an elevation equal to the latitude of the place. (3) With a reflector or a piece of white paper reflect light into the telescope so that the cross-hairs and the image of Polaris will be visible at the same time. (4) Depress the telescope and establish a target at a distance of about 500 feet; place the plank on the ground and nail it firmly to a flat stake driving one at each end. (5) Level up again and follow Polaris with the telescope by means of the tangent movement; at elongation it will appear to traverse the vertical hair for several minutes. (6) Depress the telescope, sight at a pencil held on the target and mark the point very carefully. (7) As a check make three observations within half an hour after elongation, noting the time of sighting on the star. Reverse the instrument in altitude and azimuth after the first check observation. (8) Reduce the check observations to observations at elongation by the following rule: Multiply the square of the time since elongation in minutes by 0.058, and the product will be the correction to the azimuth of Polaris in seconds of arc, for latitude 40°. (9) The next morning lay off the azimuth of Polaris for each observation to the east or west depending upon whether the observation was made at western or eastern elongation. (10) Check the observed meridian with the standard meridian. The error of the mean of the four observation should not exceed one minute. Record and reduce the data as in the prescribed form.

PROBLEM D12. DETERMINATION OF TRUE MERIDIAN BY OBSERVATION ON POLARIS AT ANY TIME.

(a) *Equipment.*—The same as in D11.

(b) *Problem.*—Determine a true meridian by observing Polaris at any time.

(c) *Methods.*—(1) Make the observations in the manner described in D11. (2) Compute the azimuth of Polaris at the time of each observation, using the tables given in the U. S. Land Survey Manual, pp. 118-119; Johnsons' Surveying, pp. 814-815; Wilsons' Topographic Surveying, pp. 716-

DETERMINATION OF TRUE MERIDIAN BY OB'S ON POLARIS AT ELONGATION.

No. Obs.	Bubble.	R.R. Time Obs.	Azimuth of Polaris Observed	Azimuth of Polaris Correction	Azimuth of Polaris Elong'tn	Error Azimuth
1	Down	2h 17m	1°35.0'	0°0.0'	1°35.0'	− 0.7'
2	"	2 30	1°35.5'	0°0.1'	1°35.6'	−0.1'
3	Up	2 40	1°36.5'	0°0.5'	1°37.0'	+1.3'
4	"	2 43	1°36.5'	0°1.0'	1°37.5'	+1.8'
			Mean		1°36.4'	+0.7'
			(Allowable Error			1.0')

Calculation of Railroad Time of Elongation.

Latitude 40°06' Longitude 88°15'

	h. m.
Astron. Time U.C. Polaris, Dec. 1. 1899	8 38.8
Reduction for 3 days is 3 × 3.94m	− 11.8
Astron. Time U.C. Polaris, Dec. 4. 1899	8 27.0
Correction for Railroad Time	− 7.0
R.R. Time U.C. Polaris, Dec. 4, 1899	8 20.0
Reduction for Western Elongation	+5 57.0
Railroad Time " "	2h 17m A.M.

Calculation of Azimuth of Polaris at Elong'tn

Azimuth Polaris, Elong'tn, Jan. 1, 1899	1°36.4'
Correction for Dec. 4th. 1899	− 0.7
Azimuth Polaris, Elong'th Dec. 4, '99	1°35.7'

Reduction Sidereal Time Elongation-te

$\cos t_e^{\circ} = \tan \phi, \cot \delta = \tan 40°06', \cot 88°45'$

$t_e = 5^{h.} 56^{m}$

t_e reduced to RR Time $= 5^{h.} 57^{m}$

Observers { J. Doe. R. Roe.

Dec. 4, 1899 (2 hours), Clear and Warm.

Buff & Berger Transit No. 9, 2 Lanterns, hubs, 2 flat stakes, plank 18"×4"×2", 4-8d nails. Axe, Watch set to keep Railroad Time.

Set Transit over hub at 1-40 A.M., sighted at Polaris, depressed the telescope and established target about 500 ft. from instrument; the plank was placed at right angles to line and nailed to a stake driven at each end.

Made first observation at western elongation

Reversed instrument in altitude and azimuth between 2nd & 3d readings.

Reduced observations 2, 3 & 4 by the following formula:—

Corr" = 0.058 t^2

where t = time from elongation in minutes; the correction being in seconds of arc.

(For Latitude 40°; 30 min. from elongation)

Polaris at Upper Culmination.

AZIMUTHS OF POLARIS AT ELONGATION

Between 1900 and 1910 and Latitudes 30 and 60° North.

(From U. S. Land Survey Manual.)

Latitude.	1900.	1901.	1902.	1903.	1904.	1905.
°	° ′	° ′	° ′	° ′	° ′	° ′
30	1 24.9	1 24.6	1 24.2	1 23.9	1 23.5	1 23.1
31	25.8	25.5	25.1	24.7	24.4	24.0
32	26.7	26.4	26.0	25.6	25.3	24.9
33	27.7	27.3	27.0	26.6	26.2	25.9
34	28.7	28.4	28.0	27.6	27.2	26.9
35	1 29.8	1 29.4	1 29.0	1 28.7	1 28.3	1 27.9
36	30.9	30.5	30.1	29.8	29.4	29.0
37	32.1	31.7	31.3	30.9	30.5	30.1
38	33.4	33.0	32.6	32.2	31.8	31.4
39	34.7	34.3	33.9	33.5	33.1	32.7
40	1 36.0	1 35.6	1 35.2	1 34.8	1 34.4	1 34.0
41	37.5	37.1	36.7	36.2	35.8	35.4
42	39.0	38.6	38.2	37.7	37.3	36.9
43	40.6	40.2	39.8	39.3	38.9	38.5
44	42.3	41.8	41.4	41.0	40.5	40.1
45	1 44.0	1 43.6	1 43.2	1 42.7	1 42.3	1 41.8
46	45.9	45.5	45.0	44.6	44.2	43.7
47	47.9	47.4	46.9	46.5	46.0	45.6
48	49.9	49.5	49.0	48.6	48.1	47.7
49	52.1	51.7	51.2	50.7	50.2	49.8
50	1 54.4	1 54.0	1 53.5	1 53.0	1 52.5	1 52.0

Latitude.	1906.	1907.	1908.	1909.	1910.
°	° ′	° ′	° ′	° ′	° ′
30	1 22.8	1 22.4	1 22.1	1 21.7	1 21.3
31	23.6	23.2	22.9	22.5	22.2
32	24.5	24.1	23.8	23.4	23.1
33	25.5	25.1	24.7	24.3	24.0
34	26.5	26.1	25.7	25.3	25.0
35	1 27.5	1 27.1	1 26.8	1 26.4	1 26.0
36	28.6	28.2	27.9	27.5	27.1
37	29.7	29.3	29.0	28.6	28.2
38	31.0	30.6	30.2	29.8	29.4
39	32.3	31.8	31.4	31.0	30.6
40	1 33.6	1 33.2	1 32.8	1 32.4	1 32.0
41	35.0	34.6	34.2	33.8	33.4
42	36.5	36.0	35.6	35.2	34.8
43	38.1	37.6	37.2	36.8	36.3
44	39.7	39.2	38.8	38.4	37.9
45	1 41.4	1 40.9	1 40.5	1 40.1	1 39.6
46	43.2	42.7	42.3	41.9	41.4
47	45.1	44.6	44.2	43.7	43.3
48	47.2	46.7	46.3	45.8	45.3
49	49.3	48.8	48.4	47.9	47.4
50	1 51.5	1 51.0	1 50.6	1 50.1	1 49.6

CORRECTION TO AZIMUTHS OF POLARIS FOR EACH MONTH.

(From U. S. Land Survey Manual.)

For middle of—	Latitude.			For middle of—	Latitude.		
	25°.	40°.	55°.		25°.	40°.	55°.
	′	′	′		′	′	′
January	− 0.3	− 0.4	− 0.5	July........	+ 0.2	+ 0.3	+ 0.4
February ...	− 0.3	− 0.3	− 0.4	August.....	+ 0.1	+ 0.1	+ 0.2
March......	− 0.1	− 0.2	− 0.2	September..	0.0	− 0.1	− 0.1
April.......	0.0	0.0	0.0	October	− 0.2	− 0.3	− 0.4
May........	+ 0.2	+ 0.2	+ 0.2	November..	− 0.5	− 0.6	− 0.7
June........	+ 0.2	+ 0.3	+ 0.4	December...	− 0.6	− 0.8	− 0.9

LOCAL MEAN TIME OF UPPER CULMINATION OF POLARIS.

Computed for Longitude 6 hours or 90° W. of Greenwich.

(From U. S. Land Survey Manual.)

Date.	1900	1901.	1902.	1903.	1904.	1905.	Diff. for 1 Day.
	h. m.	h. m.	h. m.	h. m.	h. m.	h. m.	m.
Jan. 1	6 36.3	6 37.4	6 38.5	6 39.6	6 40.7	6 41.8	3.95
15	5 41.0	5 42.1	5 43.2	5 44.3	5 45.4	5 46.5	3.95
Feb. 1	4 33.9	4 35.0	4 36.1	4 37.2	4 38.3	4 39.4	3.95
15	3 38.6	3 39.7	3 40.8	3 41.9	3 43.0	3 44.1	3.95
Mar. 1	2 43.4	2 44.5	2 45.6	2 46.7	2 47.8	2 48.9	3.94
15	1 48.2	1 49.3	1 50.4	1 51.5	1 52.6	1 53.7	3.94
Apr. 1	0 41.3	0 42.4	0 43.5	0 44.6	0 45.7	0 46.8	3.94
15	23 42.4	23 43.5	23 44.6	23 45.7	23 46.8	23 47.9	3.93
May 1	22 39.5	22 40.6	22 41.7	22 42.8	22 43.9	22 44.0	3.93
15	21 44.6	21 45.7	21 46.8	21 47.9	21 49.0	21 50.1	3.92
June 1	20 38.0	20 39.1	20 40.2	20 41.3	20 42.4	20 43.5	3.92
15	19 43.2	19 44.3	19 45.4	19 46.5	19 47.6	19 48.7	3.92
July 1	18 40.5	18 41.6	18 42.7	18 43.8	18 44.9	18 46.0	3.92
15	17 45.7	17 46.8	17 47.9	17 49.0	17 50.1	17 51.2	3.92
Aug. 1	16 39.1	16 40.2	16 41.3	16 42.4	16 43.5	16 44.6	3.91
15	15 44.3	15 45.4	15 46.5	15 47.6	15 48.7	15 49.8	3.92
Sept. 1	14 37.6	14 38.7	14 39.8	14 40.9	14 42.0	14 43.1	3.92
15	13 42.7	13 43.8	13 44.9	13 46.0	13 47.1	13 48.2	3.92
Oct. 1	12 39.9	12 41.0	12 42.1	12 43.2	12 44.3	12 45.4	3.93
15	11 44.9	11 46.0	11 47.1	11 48.2	11 49.3	11 50.4	3.93
Nov. 1	10 38.1	10 39.2	10 40.3	10 41.4	10 42.5	10 43.6	3.93
15	9 42.9	9 44.0	9 45.1	9 46.2	9 47.3	9 48.4	3.94
Dec. 1	8 39.9	8 41.0	8 42.1	8 43.2	8 44.3	8 45.4	3.94
15	7 44.7	7 45.8	7 46.9	7 48.0	7 49.1	7 50.2	3.94

717. (3) The next morning lay off the computed azimuth for each observation. (4) Check the observed meridian with the standard meridian. The error of the mean of the five observations should not exceed one minute. Record the data.

PROBLEM D13. COMPARISON OF TRANSIT TELESCOPES.

(a) *Equipment.*—Five engineers' transits.

(b) *Problem.*—Make a critical comparison of the telescopes of five engineers' transits.

(c) *Methods.*—Follow the methods outlined in the comparison of level telescopes.

PROBLEM D14. TEST OF A TRANSIT.

(a) *Equipment.*—Transit, reading glass, leveling rod, chaining pins, foot rule.

(b) *Problem.*—Test the following adjustments of an assigned transit: (1) Test the graduation for eccentricity. (2) Test the plate levels to see if they are perpendicular to the vertical axis. (3) Test the line of collimation to see if it is perpendicular to the horizontal axis. (4) Test the horizontal axis to see if it is perpendicular to the vertical axis. (5) Test the level under the telescope to see if the tangent to the tube at the center is parallel to the line of collimation. (6) Test the vertical circle to see if the vernier reads zero when the line of sight is horizontal.

(c) *Methods.*—Make the tests as described in the first part of this chapter but do not make any of the adjustments or tamper with any of the parts of the instrument. Check each test. Make a careful record of the methods and errors, including a statement of the manner of doing correct work with each adjustment out.

PROBLEM D15. ADJUSTMENT OF A TRANSIT.

(a) *Equipment.*—Transit, reading glass, leveling rod, chaining pins, adjusting pin, small screw driver.

(c) *Methods.*—Make the following tests and adjustments of an assigned transit that has been thrown out of adjustment by the instructor: (1) Test the graduation for eccentricity. (2) Adjust the plate levels perpendicular to the vertical axis. (3) Adjust the line of collimation perpendicular to the vertical axis. (4) Adjust the horizontal axis perpendicular to the vertical axis. (5) Adjust the level under the telescope parallel to the line of collimation. (6) Adjust the zero of the vertical circle to read zero when the line of sight is horizontal. (7) Center the eyepiece.

(c) *Methods.*—Make the tests and adjustments as described in the first part of this chapter. Use extreme care in manipulating the screws and if any of the parts stick or work harshly, call the instructor's attention before proceeding. Repeat the tests and adjustments. Make a careful record of methods and errors.

PROBLEM D16. SKETCHING A TRANSIT.

(a) *Equipment.*—Engineers' transit.

(b) *Problem.*—Make a first-class sketch of an engineers' transit.

(c) *Methods.*—(See similar problem with the level.)

ERROR OF SETTING FLAG POLE WITH ENGINEERS' TRANSIT.

Distance Ft.	No. of Setting	Distance In.	d In	d^2	
300	1	1.18	0.16	0.0324	
	2	1.38	.02	.0004	
	3	1.30	.06	.0036	
	4	1.53	.17	.0289	
	5	1.32	.04	.0016	
	6	1.38	.02	.0004	
	7	1.28	.07	.0049	
	8	1.45	.10	.0100	
	9	1.46	.10	.0100	
	10	1.30	.06	.0036	
	Mean	1.36		0.0958	$=\Sigma d^2$
600	1	1.14	0.25	0.0625	
	2	1.56	.17	.0289	
	3	1.14	.25	.0625	
	4	1.22	.17	.0289	
	5	1.76	.37	.1369	
	6	1.55	.16	.0256	
	7	1.23	.16	.0256	
	8	1.10	.28	.0841	
	9	1.55	.16	.0256	
	10	1.65	.26	.0676	
	Mean	1.39		0.5472	$=\Sigma d^2$

Observers { J. Doe. R. Roe.

Dec 6, 1888, (2 hours). Cool and quiet.
Used Buff & Berger Transit, Locker No. 8, flat stake, 1″×2″×15″, and iron flag pole.
Sighted at iron flag pole set on stake which had been placed on ground at about 300 ft. from the Transit, and clamped both plates; then measured the distance in inches from a line drawn across the board.
With both plates clamped lined in the rod 10 times in all, the flagman noting the distance from the line. The pole was shifted each time.
Repeated test for 600 ft.

Probable Error for 300 Ft.

$$E_1 = 0.6745\sqrt{\frac{\Sigma d^2}{n-1}} = 0.6745\sqrt{\frac{.0958}{9}} = 0.103 \text{ in.}$$

$$E_m = \frac{E_1}{\sqrt{n}} = \frac{0.103}{\sqrt{10}} = 0.032 \text{ in.} = 0.0027 \text{ ft}$$

$$E_m \text{ (Angle)} = \tan^{-1}\frac{0.0027}{300} = 1.8''$$

Probable Error for 600 ft.

$$E_1 = 0.6745\sqrt{\frac{.5472}{9}} = 0.247 \text{ in.}$$

$$E_m = \frac{0.247}{\sqrt{10}} = 0.078 \text{ in.} = 0.0065 \text{ ft.}$$

$$E_m \text{ (Angle)} = \tan^{-1}\frac{0.0065}{600} = 2.2''$$

PROBLEM D17. ERROR OF SETTING FLAG POLE WITH TRANSIT.

(a) *Equipment.*—Transit, iron flag pole, flat stake 1″x 2″x 15″, foot rule.

(b)*Problem.*—Determine the probable error of setting a flag pole with the transit at a distance of 300 feet. Repeat for 600 feet.

(c) *Methods.*—(1) Set the transit up and sight at the flag pole plumbed near the middle of the stake at a distance of about 300 feet. (2) Measure the distance from the point of the flag pole to a mark on the stake. (3) Keep the vertical axis clamped, and move the pole to one side. (4) Set the pole with the transit, and measure the distance from the first line. (5) Repeat until at least ten consecutive satisfactory results are obtained. (6) Compute the probable error of a single observation and of the mean of all the observations (see chapter on errors of surveying), and reduce the mean error to its angular value. (7) Repeat for 600 feet. Determine distances by pacing. Follow the prescribed form.

PROBLEM D18. REPORT ON DIFFERENT MAKES AND AND TYPES OF TRANSITS.

(a) *Equipment.*—Department equipment, catalogs of the principal makers of engineers' transits.

(b) *Problem.*—Make a critical comparison of the several types of transits made by the different makers.

(c) *Methods.*—(See similar problem with the level.)

CHAPTER VI.

TOPOGRAPHIC SURVEYING.

Topographic Map.—A topographic map is one which shows with practical accuracy all the drainage, culture, and relief features that the scale of the map will permit. These features may be grouped under three heads as follows: (1) the culture, or features constructed by man, as cities, villages, roads; (2) the hypsography, or relief of surface forms, as hills, valleys, plains; (3) the hydrography, or water features, as ponds, streams, lakes. The culture is usually represented by conventional symbols. The surface forms are shown by contours (lines of equal height), (a) Fig. 24, or hachures, (b) Fig. 24. The water features are shown by soundings, conventional signs for bars, etc.

Topographic maps may be divided into two classes depending upon the scale of the map. Small scale topographic maps are made by the U. S. Coast and Geodetic Survey and the U. S. Geological Survey, and are drawn to a scale of 1:62,500, 1:125,000 or 1:250,000 with corresponding contour intervals of 5 to 50, 10 to 100, and 200 to 250 feet. These maps show the streams, highways, railroads, canals, etc., in outline but do not show any features of a temporary character.

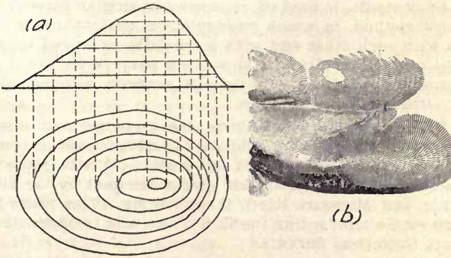


Fig. 24.

Large scale topographic maps are drawn to a scale of 400 feet to 1 inch (1:4800), or greater, with contour intervals from 1 to 10 feet depending upon whether the ground is flat or hilly. Roads, streets, dwellings, streams, etc., are drawn to scale. Features too small to be properly represented when drawn to scale are drawn out of proportion to the scale of the map.

Topographic Survey.—The object of a topographic survey is the production of a topographic map, and hence neither time nor money should be wastefully expended in obtaining field data more refined than the needs of the mapping demand.

METHODS.—A topographic survey may be divided into three parts: (1) the reconnaissance; (2) the skeleton of the survey; (3) filling in the details.

Reconnaissance.—The reconnaissance is a rapid preliminary survey to determine the best methods to use in making the survey and the location of the principal points of control. A careful reconnaissance enables the topographer to choose methods that are certain to result in a better map and a distinct saving of time.

Skeleton.—There are three general methods of locating the skeleton of a topographic survey: (1) tie line survey with chain only; (2) traverse method with transit or compass; (3) triangulation system, (f), Fig. 30. The first method is used for the survey of small tracts. The second method, in which the distances are measured with the chain, tape or stadia, is used on railroad and similar surveys. The third method, in which triangulation stations are connected with each other and with a carefully measured base line and base of verification, is used on surveys for small scale maps and on detailed or special surveys, such as surveys of cities and reservoir sites.

Filling in Details.—There are three general methods employed for filling in details: (1) with transit or compass and chain; (2) with transit and stadia; (3) with plane table and stadia. The transit and stadia are used by the Mississippi and Missouri River Commissions. The plane table and stadia are used by the U. S. Coast and Geodetic and the U. S. Geological Surveys.

Topographic City Survey.—A topographic city survey is one of the best examples of a survey for a large scale map.

It is usually based on a system of triangulation executed with precision and connected with carefully measured base lines. The details of the survey are usually taken up in the following order: (1) reconnaissance and location of triangulation stations; (2) measurement of base line and base of verification; (3) measurement of angles by repetition; (4) establishment of bench marks by running duplicate levels; (5) adjustment of angles of triangulation system; (6) computation of sides, azimuths and coordinates; (7) filling in details, usually with transit and stadia; (8) plotting of triangulation and other important points on the map by rectangular coordinates; (9) plotting the details and completing the map. The instructions given on the succeeding pages are for a survey of this type.

HYDROGRAPHIC SURVEY.

Classes.—Hydrographic surveying may be divided into river and marine. The first includes the determination of depths, location of bars, and obstructions to navigation, determination of area of cross-section, discharge, sediment carried, etc. The second includes the making of soundings, location of bars, ledges, buoys, etc. The depth of the water is determined by making soundings with a lead or rod, and the velocity is gaged by means of floats or a current meter, (d), Fig. 31.

Soundings are located: (1) by two angles read simultaneously from both ends of a line on the shore, (f), Fig. 31; (2) by keeping the boat in line with two flags on shore, and determining the position on the line by means of an angle read on the shore, or by a time interval; (3) by intersecting ranges, (g), Fig. 31; (4) by stretching a rope or wire across the stream; (5) by measuring with a sextant in the boat at the instant that the sounding is taken two angles to three known points on the shore, (c), Fig. 31; the point is located by solving the three point problem graphically with the three arm protroctor, (e), Fig. 31; (6) by locating the position of the boat at the instant that the soundings are taken with transit and stadia. The first three methods are used on small river or lake surveys. The fourth method is used where soundings are taken at frequent intervals. The fifth method has been used almost exclusively in locating soundings in harbors, lakes, and large rivers. The sixth method is rapidly coming into general use and promises to be the favorite method.

THE STADIA.

Description.—The stadia is a device for measuring distances by reading an intercept on a graduated rod. The stadia-hairs, shown in (g), Fig. 27, are carried on the same reticule as the cross-hairs and are placed equidistant from the horizontal hair. The stadia-hairs are sometimes placed on a separate reticule and made adjustable. It is, however, considered better practice by most engineers to have the stadia-hairs fixed and use an interval factor, rather than try to space the hairs to suit a rod or to graduate a rod to suit an interval factor.

Stadia Rods.—Stadia rods are always of the self reading type. In Fig. 27, (a) and (b) are the kind used on the U. S. Coast Survey; (c) on the U. S. Lake Survey; (d) and (c) by the U. S. Engineers. A target for marking on the rod the height of the horizontal axis of the transit above the station occupied is shown in (f), Fig. 27.

Theory of the Stadia.—In Fig. 25, by the principles of optics, rays of light passing from points A and B on the rod through the objective so as to emerge parallel and pass through the stadia-hairs a and b, respectively, must intersect at the principal focal point d in front of the objective; therefore the rod intercept, *s* is proportional to the distance, *g* from the principal focal point in front of the objective.

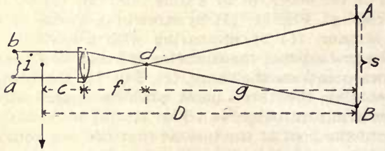


Fig. 25.

Stadia Formula For Horizontal Line of Sight and Vertical Rod.—In Fig. 25, from similar triangles we have

$$s : g :: i : f \qquad (1)$$

From which

$$g = \frac{f}{i}\ s = k\,s \qquad (2)$$

and

$$D = k\,s + (c + f) \qquad (3)$$

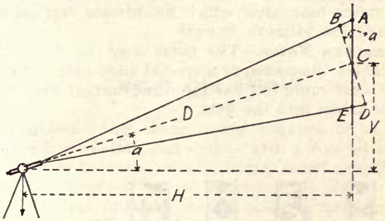


Fig. 26

Stadia Formula For Inclined Line of Sight and Vertical Rod.—In Fig. 26 we have

$$BD = AE \cos a \quad \text{(approx.)} \qquad (4)$$

and

$$D = k\,s \cos a + (c + f) \qquad (5)$$

but

$$H = D \cos a \qquad (6)$$

$$= k\,s \cos^2 a + (c + f) \cos a \qquad (7)$$

$$= k\,s - k\,s \sin^2 a + (c + f) \cos a \qquad (8)$$

also

$$V = D \sin a \qquad (9)$$

$$= k\,s \sin a \cos a + (c + f) \sin a \qquad (10)$$

$$= \tfrac{1}{2} k\,s \sin 2a + (c + f) \sin a \qquad (11)$$

USE OF THE STADIA.—The transit is set up over a station of known elevation and with a given direction or azimuth to another visible station; the height of the line of collimation above the top of the station is determined either by holding the rod beside the instrument and setting the target, or preferably by graduating one leg of the tripod and using the plumb bob; then with the transit oriented on a given line, "shots" are taken to representative points, and record made of the rod intercept, vertical angle and azimuth. In reading the intercept the middle hair is first set roughly on the target, then one stadia-hair is set at the nearest foot-mark on the rod and the intercept read with the other stadia-hair, after which the precise vertical angle is taken, and the azimuth is read.

Reducing the Notes.—The notes may be reduced by means of tables, diagrams, or a special slide rule. The slide rule is the most rapid but has the disadvantage that it cannot well be taken into the field.

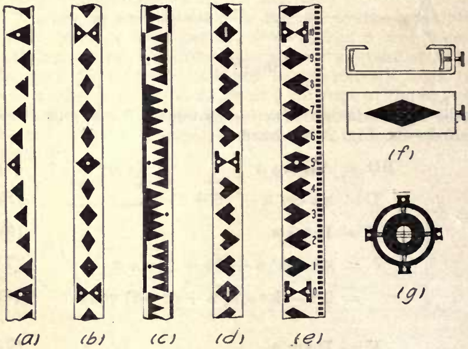


Fig. 27.

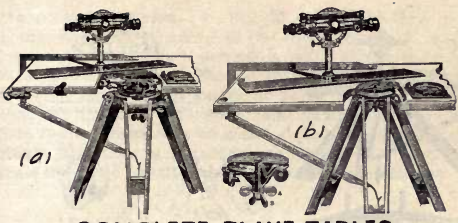


Fig. 28.

THE PLANE TABLE.

Description.—The plane table consists of an alidade, carrying a line of sight and a ruler with a fiducial edge. The alidade is free to move on a drawing board mounted on a tripod. The drawing board is leveled by means of plate levels. The line of sight should make a fixed horizontal angle with the fiducial edge of the ruler. The complete plane table is a transit in which the horizontal limb has been replaced by a drawing board.

There are three general types of plane tables: (1) the Coast Survey plane table, (a) Fig. 28; (2) the Johnson plane table, (b), Fig. 28; (3) the Gannet plane table, (d), Fig. 29.

USE OF THE PLANE TABLE.—In making a survey with a plane table the angles are measured graphically and the lines and points are plotted in the field. The principal methods of making a survey with a plane table are: (1) radiation; (2) traversing; (3) intersection; (4) resection.

Radiation.—In this method a convenient point on the paper is set over a selected point in the field, and the table clamped. The line of sight is then directed towards each point to be located in turn and a line is drawn along the fiducial edge of the ruler. The distances, which may be determined by measuring with chain, tape or stadia, are plotted to a convenient scale, (a), Fig. 30.

Traversing.—This method is practically the same as trav-

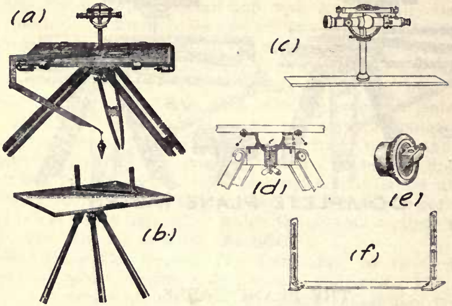


Fig. 29.

ersing with a transit,, (b), Fig. 30. Care should be used in orienting the plane table to get the point on the paper over the corresponding point on the ground as nearly as the character of the work requires.

Intersection.—In this method the points are located by intersecting lines drawn from the ends of a measured base line, (c), Fig. 30.

Resection.—In the resection method the plane table is set up at a random point and oriented with respect to either three or two given points, which gives rise to two methods known respectively as the three-point and two-point problems.

Three Point Problem.—Where three points are located on the map and are visible but inaccessible, the plane table is oriented by solving the "three point problem". There are several solutions, the best known of which are: (1) the mechanical solution; (2) the Coast Survey solution; (3) Bessel's solution; (4) analytical solution. The problem is indeterminate if a circle can be passed through the four points.

In the mechanical solution the two angles subtended by the three points are plotted graphically on a piece of tracing paper and the point is located by placing the tracing paper over the plotted points.

In Bessell's solution, (d), Fig. 30, a, b, c are three points on the map corresponding to the three points, A, B, C on the ground, and D is the random point at the instrument whose location, d, it is desired to find on the map. Construct the angle 1 with vertex at point c as follows: Sight along the line ca at the point C, and clamp the vertical axis. Then center the alidade on c and sight at B by moving the alidade, and draw a line along the edge of the ruler. Construct the angle 2 with vertex at a in the same manner. The

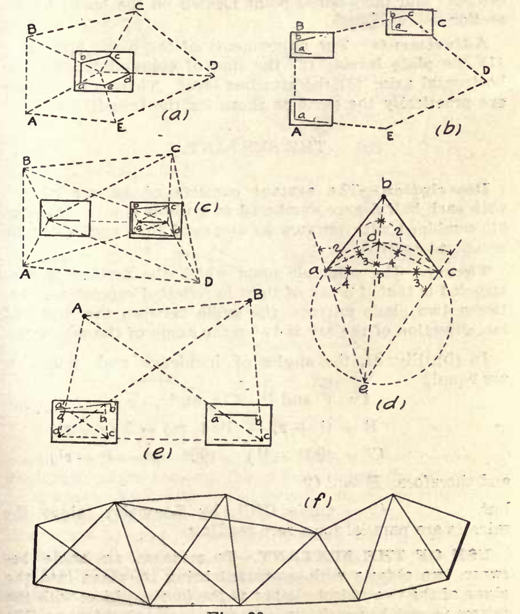


Fig. 30.

line joining b and e will pass through the point d required. Orient the board by sighting at B with the line of sight along the line e b, and locate d by resection.

Two Point Problem.—To orient the board when only two points are plotted, proceed as follows: Select a fourth point, c, that is visible, and with these two points as the ends of a base line, (e), Fig. 30, laid off to a convenient scale, locate two points a′ and b′ on the map by intersection. The error of orienting the board will be the angle between the lines a, b and a′ b′. The table can now be oriented and the desired point located on the board by resection.

Adjustments.—The adjustments of the plane table are: (1) the plate levels; (2) the line of collimation; (3) the horizontal axis; (4) the attached level. These adjustments are practically the same as those for the transit.

THE SEXTANT.

Description.—The sextant consists of an arc of 60°, with each half degree numbered as a whole degree, (a), Fig. 31, combined with mirrors so arranged that angles can be measured to 120°.

Theory.—The principle upon which the sextant is constructed is that if a ray of light is reflected successively between two plane mirrors, the angle between the first and last direction of the ray is twice the angle of the mirrors.

In (b), Fig. 31, the angles of incidence and reflection are equal,

$$i = r \text{ and } i^1 = r^1, \text{ and}$$

$$E = (i + r) - (i^1 + r^1) = 2\,(r - r^1)$$

$$C^1 = (90^\circ - i^1) - (90^\circ - r) = (r - r^1)$$

and therefore $E = 2\,C^1$

but C^1 = angle CIC^1, by Geometry, since the mirrors are parallel for a zero reading.

USE OF THE SEXTANT.—To measure an angle between two objects with a sextant bring its plane into the plane of the two objects; sight at the fainter object with the telescope and bring the two images into coincidence. The

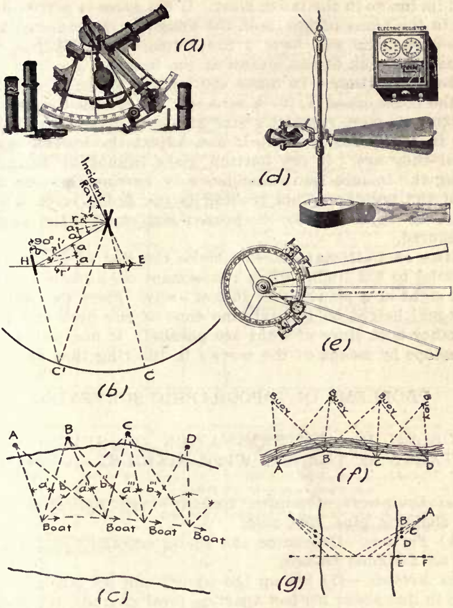


Fig. 31.

reading is the angle sought. The angle will not be the true horizontal angle between the objects unless the objects are in the same level with the observer. Since the true vertex of the measured angle shifts for different angles the sextant should not be used for measuring small angles between objects near at hand.

ADJUSTMENTS. Index Glass,—To make the index glass, I, perpendicular to the plane of the limb, bring the vernier to about the middle of the arc and examine the arc and its image in the index glass. If the glass is perpendicular to the plane of the limb, the image of the reflected and direct portions will form a continuous curve. Adjust the glass by means of the screws at the base.

Horizon Glass.—To make the horizon glass, H, parallel to the index glass, I, for a zero reading. With the vernier set to read zero, sight at a star and note if the two images are in exact coincidence. If not, adjust the horizon glass until they are. If the horizon glass cannot be adjusted, bring the images into coincidence by moving the arm and read the vernier. This reading is the index error which must be applied with its proper sign to all the angles measured.

Line of Collimation.—To make the line of collimation parallel to the limb. Place the sextant on a plane surface and sight at a point about 20 feet away. Place two objects of equal height on the extreme ends of the limb and note whether both lines of sight are parallel. If not, adjust the telescope by means of the screws in the ring that carry it.

PROBLEMS IN TOPOGRAPHIC SURVEYING.

PROBLEM E1. DETERMINATION OF STADIA CONSTANTS OF TRANSIT WITH FIXED STADIA-HAIRS.

(a) *Equipment.*—Complete transit, stadia rod. steel tape, set chaining pins, foot rule.

(b) *Problem.*—Determine the stadia constants c, f and k for an assigned transit.

(c) *Methods.*—(1) Set up the transit and set ten chaining pins in line about 100 feet apart on level ground. (2) Plumb the stadia rod by the side of the first pin. (3) Set the lower hair on an even foot or half foot mark keeping the telescope nearly level, and read the upper stadia-hair. (4) Record the intercept. (5) Read the intercept on the rod at the remaining pins. (6) Measure the distance from the center of the transit to each pin with the steel tape. (7) Focus the objective on a distant object, measure f (the distance from the plane of the cross-hairs to the center of the objective), and c(the distance from the center of the objective to the center of the instrument). (8) Calculate the value of the stadia ratio, k, for each distance by substituting in the

Observers—{ J. Doe. / R. Roe.

DETERMINATION OF STADIA CONSTANTS — FIXED HAIRS.

No	S Ft.	D Ft	D-(c+f) (c+f=1.17ft)	K (Ft.)	d	d^2
1	1.81	180.41	179.24	99.02	0.02	0.0004
2	2.70	268.40	267.23	98.96	0.08	0.0064
3	3.58	355.32	354.15	98.92	0.12	0.0144
4	4.05	400.89	399.72	98.72	0.32	0.1024
5	4.86	482.80	481.63	99.11	0.07	0.0049
6	5.61	556.30	555.13	99.20	0.16	0.0256
7	6.50	643.58	642.41	98.84	0.20	0.0400
8	7.90	786.93	785.76	99.47	0.43	0.1849
9	9.15	914.40	913.23	98.91	0.13	0.0169
10	10.31	1024.71	1023.54	99.26	0.22	0.0484
			Mean =	99.04	$\Sigma d^2 =$	0.4443

$$E_1 = \sqrt{\frac{\Sigma d^2}{n-1}} = \sqrt{\frac{0.4443}{9}} = 0.22 \text{(ft.)}$$

$$E_m = \sqrt{\frac{E_1}{n}} = \sqrt{\frac{0.22}{10}} = 0.15 \text{(ft.)}$$

c = 0.47 ft.
f = 0.70 ft.
c+f = 1.17 ft.

Dec. 14, 1899. (2 hours) Cool and Cloudy.
Used Buff & Berger Transit, Locker No. 12, and Chaining Locker No. 38.
Set 10 chaining pins in line about 100 ft. apart on level ground.
With telescope of the Transit nearly level, determined intercept "S" at each pin by setting lower hair on a foot or half-foot mark and reading upper hair.
Measured the distance from the center of the Transit to each pin with steel tape to nearest 0.01 ft.
With object glass focused on a distant object determined c and f by measuring the distance from the center of the objective to the center of the horizontal axis and the plane of the cross wires, respectively.
Determined the different values of K by substituting in the formula
$D = Ks + c + f$.

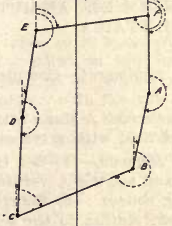
Observers { J. Doe. / R. Roe.

AZIMUTH TRAVERSE WITH TRANSIT AND STADIA.

Station Inst.	Station Obj.	Azimuth	Mag. Bearing	Distance Ft.	Vertical Angle.	Elevation Ft.
A						718.00
	F	0°00'	N4°00'W	432	+0°20'	
	B	196°13'	S12°10'W	622	−0°40'	(−7.2)
B						710.8
	A	16°13'	N12°10'E	624	+0°38'	
	C	227°16'	S43°15'W	489	+0°50'	(+7.3)
C						718.1
	B	47°16'	N43°10'E	489	−0°50'	
	D	0°03'	N4°05'W	756	−1°10'	(−15.4)
D						702.7
	C	180°03'	S4°00'E	756	+1°12'	
	E	6°14'	N2°20'E	618	+0°56'	(+10.1)
E						712.8
	D	186°14'	S2°20'W	618	−0°56'	
	F	85°46'	N85°15'E	473	+0°54'	(+7.4)
F						720.2
	E	265°46'	S85°15'W	475	−0°54'	
	A	186°01'	S4°05'E	434	−0°20'	(−2.5)
A						717.7
		180°00'				718.0
Error		01'			Error	0.3

Allowable Error 2'   Allowable Error 0.5 ft.

Dec. 15, 1899, (3 hours) Clear and Warm.
Used Buff & Berger Transit, Locker No. 12, and Stadia Board No. 6.
Stadia Constants: c+f = 1.17 ft., K = 100.00
Sighted at target set at H.I. for Vert. Angle.
Oriented the Transit by Azimuth reversals.



fundamental stadia formula. (9) Take the arithmetical mean of the ten determinations as the true value. (10) Compute the probable error of a single observation and of the mean of all the observations. The interval factor should be determined by the instrument man under the conditions of actual work. The determination should be checked at frequent intervals during the progress of the field work. Follow the prescribed form.

PROBLEM E2. STADIA REDUCTION TABLE.

(a) *Equipment.*—(No instrumental equipment required.)

(b) *Problem.*—Compute a stadia reduction table giving the horizontal distances from a point in front of the objective equal to the principal focal distance for the stadia intervals from 0.01 feet to 10 feet, for the transit used in Problem E1.

(c) *Methods.*—(1) Prepare form for calculation. (2) Compute the horizontal distances by substituting the different values of s in the stadia formula. Compute D′ for values of s varying from 0.01 foot to 0.1 foot varying by 0.01 foot; from 0.1 foot to 1 foot varying by 0.1 foot; and from 1 foot to 10 feet varying by 1 foot.

(To use the table, take the sum of the values of D′ corresponding to the units, tenths and hundredths of s as given in the table. To the value of D′ thus obtained add c plus f.)

PROBLEM E3. AZIMUTH TRAVERSE WITH TRANSIT AND STADIA.

(a) *Equipment.*—Complete transit, stadia rod, steel pocket tape.

(b) *Problem.*—Make a traverse of the perimeter of an assigned field with a transit and stadia.

(c) *Methods.*—(1) Set the transit over one corner of the field and set the A vernier to read the azimuth of the preceding course. (2) Sight at a stadia rod held edgewise on the last station to the left with the telescope normal and clamp the lower motion. (3) Read the intercept on the rod to the nearest 0.01 foot. (4) Sight at the target set at the first station and read the vertical angle to the nearest minute. (The observer should measure the height of the horizontal axis above the station with the steel pocket tape, or one tripod leg may be graduated and the instrument height determined by swinging the plumb bob out against the leg.) (5) Unclamp the upper motion, sight at the next station to the right and clamp the upper motion. (6) Read the A vernier, which will be the azimuth of the course. (7) Read the intercept on the rod. (8) Measure the vertical angle by sighting at the target set at the height of the horizontal axis as before. (9) Set the transit over the next station to the right and determine the intercepts and vertical angles as at the first station. (10) Determine the stadia intercepts and vertical angles at the remaining stations, passing around the field to the right. (11) Reduce the intercepts to horizontal distances before recording. (12) Compute the vertical differences in elevation using mean distances and vertical angles. (13) Compute latitudes and departures to the nearest foot using a traverse diagram or traverse table. Follow form B4. The angular error of closure for a six-sided field should not exceed 2′ Follow the prescribed form for the field notes.

PROBLEM E4. SURVEY OF FIELD WITH PLANE TABLE BY RADIATION.

(a) *Equipment*—Plane table, stadia rod, 2 flag poles, engineers' divided scale, drawing paper, 6H pencil.

(b) *Problem.*—Make a survey of an assigned field by radiation with the plane table.

(c) *Methods.*—(1) Set the plane table up at some convenient point in the field and select a point on the drawing board that will allow the entire field to be plotted on the paper. (2) Sight at one of the stations with the ruler centered on the point on the paper. (3) Draw a line along the fiducial edge of the ruler towards the point. (4) Measure the distance to the point with the stadia. (5) Lay off the distance on the paper to the prescribed scale. (6) Locate the remaining points in the same manner. (7) Complete the map in pencil. The map should have a neat title, scale, meridian, etc. (8) Trace the map on tracing linen. (9) Compute the area by the perpendicular method, scaling the dimensions from the map.

PROBLEM E5. SURVEY OF A FIELD WITH PLANE TABLE BY TRAVERSING.

(a) *Equipment*—Plane table, stadia rod, 2 flag poles, engineers' divided scale, drawing paper, 6H pencil.

(b) *Problem.*—Make a survey of an assigned field by traversing with the plane table.

(c) *Methods.*—Follow the same general methods as those given for traversing with the transit. Adjust the plane table before beginning the problem. Complete the map and compute the area as in Problem E4.

PROBLEM E6. SURVEY OF FIELD WITH PLANE TABLE BY INTERSECTION.

(a) *Equipment.*—Plane table, 2 flag poles, engineers' divided scale, drawing paper, 6H pencil.

(b) *Problem.*—Make a survey of an assigned field with the plane table by intersection.

(c) *Methods.*—(1) Select and measure a base line having both ends visible from all the stations in the field. (2) Set the plane table over one end of the base line and sight at the other end of the base line and at each one of the stations of the field. (3) Set the plane table over the other end of the base line, orient the instrument by sighting at the station first occupied and sight at all the stations in the field. (4) Complete the map and compute the area as in Problem E4.

PROBLEM E7. THREE POINT PROBLEM WITH PLANE TABLE.

(a) *Equipment.*—Plane table, 2 flag poles, engineers' divided scale, 6H pencil.

(b) *Problem.*—Having three points plotted on the map, required to locate a fourth point on the map by solving the "three point problem" with the plane table.

(c) *Methods.*—(1) Use Bessell's solution. (2) Check by using the mechanical solution.

PROBLEM E8. ANGLES OF TRIANGLE WITH SEXTANT.

(a) *Equipment.*—Sextant, 2 flag poles.

(b) *Problem.*—Measure the angles of an assigned triangle with the sextant.

(c) *Methods.*—(1) Set the flag poles behind the monuments at two of the vertices of the triangle and stand on the monument at the third. (2) Hold the plane of the sextant horizontal, sight at one flag pole directly with the telescope and bring the image of the other flag pole into coincidence by moving the arm. (3) Read the vernier. This reading is the angle sought. (4) Repeat the measurement with the sextant inverted. Take the mean of the two readings, which should not differ more than 2′ as the true value of the angle. (5) Measure the other angles in the same manner. The error of closure should not exceed 3′. Record the data in a suitable form.

PROBLEM E9. DETERMINATION OF COEFFICIENTS OF A TAPE.

(a) *Equipment.*—Steel tape, spring balance, 2 thermometers, steel rule, 2 stout stakes, axe, 2 pieces sheet zinc 2 by 2 inches.

(b) *Problem.*—Determine the coefficients of expansion, stretch and sag for an assigned tape. Make three determinations of each and take the arithmetrical mean as the true value.

(*Standard Tapes.*—In laying off a standard or measuring a base line where a high degree of precision is required it is important that all measurements be referred to the same standard. The Bureau of Weights and Measures of the U. S. Coast and Geodetic Survey, Washington, D. C., will compare a tape with the government standard for a small fee. The tape tested is certified to be of a given length for a given temperature and pull. For example the standard tape marked "U. S. W. & M. 215" used in laying off the 100-ft. standard in Problem A. 23, was certified to be 99.9967 feet long at a temperature of 62° F. and a pull of 12 pounds when tested on a plane surface. The coefficient of expansion of this tape was 0.0000061 per degree F.)

(c) *Methods.*—(1) *Correction for Expansion.*—Measure the length of the tape on a plane surface at two different temperatures but with a constant pull determined by a spring balance. Then substitute the lengths, l and L, and temperatures, t and T in the formula

$$l - L = e(t - T)l$$

where e is the coefficient of expansion. Repeat the test and obtain three values of the coefficient e. As large a range of temperatures as possible should be secured. Take the arithmetical mean of the three determinations as the true value.

(2) *Correction for Stretch.*—Measure the length of the tape on a plane surface with two different pulls but at a constant temperature. Determine the pull with a spring balance. Then substitute the lengths l and L and the pulls p and P in the formula

$$l - L = s(p - P)l$$

where s is the coefficient of stretch. Repeat the test and obtain three values of the coefficient s. The pulls should range from 10 to 40 pounds. Take the arithmetical mean of the three determinations as the true value.

(3) *Correction for Sag.*—Remove the handles from the tape and determine its weight very carefully. Divide the weight by the length to obtain the weight per foot, w. Drive two stout hubs a little less than 100 feet apart and fasten a piece of sheet zinc with a line ruled at right angles to the line on the top of each stake. With a pull of 10 pounds, as determined by the spring balance, measure the distance between the stakes. Calculate the correction for sag by substituting the lengths l and L, pull p, and weight per foot w, in the formula.

$$l - L = \frac{l}{24}\left(\frac{wl}{p}\right)^2$$

Repeat the measurements using a pull of 20 and 30 pounds, respectively. Add the corrections for sag to each measurement and compare the results. The temperature should remain constant during the tests. To remove the possibility of an error due to temperature, observe the temperature at the time of each observation and correct the observed length for expansion before substituting in the formula.

Report the methods, data, computations and results on a suitable form.

(a) *Equipment.*—Standard tape, transit or level, stakes

PROBLEM E10. MEASUREMENT OF BASE LINE.

(number and size to be specified by instructor), axe, spring balance, 2 thermometers, lath stakes, 8-d nails, steel rule, pieces sheet zinc 2 by 2 inches.

(b) *Problem.*—Measure an assigned base line with a standard tape. Support the tape at intervals of 20 feet and note the pull and temperature. Make at least three determinations of the length of the base line. Reduce the observed results to the standard by making corrections for standard, expansion, sag, stretch and slope. Take the arithmetical mean of all the determinations as the true value.

(c) *Methods.*—(1) Set the transit over one end of the base line, sight at the other end and determine the difference in elevation and grade. (2) Drive stout square stakes to grade by "shooting" them in with the instrument in true line a little less than a full tape length apart. The tops of the lowest stake should not be less than 6 inches above the ground. (3) Fasten a piece of sheet zinc with a fine line ruled at right angles to the direction of the base line on the top of each stake. (4) Drive lath stakes in line about 20 feet apart. (5) Drive an 8-d nail through each lath stake at grade to support the tape. (6) Measure from stake to stake, the men working as follows: No. 1 plumbs up from the rear monument or holds the zero on the mark on the rear stake; No. 2 takes the spring balance and puts a pull of 16 pounds on the tape; No. 3 reads the tape and measures the fraction of a tenth with a steel rule to 0.001 feet; No. 4 records the reading of the tape and reads the two thermometers placed at the quarter points of the tape. (7) Obtain at least three determinations of the length of the base line. (8) Correct each measurement of the base for standard, expansion, sag, stretch, and slope (see problem on coefficients of a tape). The three measurements should not differ more than 1:100,000. Report methods, computations and results on a suitable form.

PROBLEM E11. CALCULATION OF TRIANGULATION SYSTEM.

(a) *Equipment*—Seven-place table of logarithms.

(b) *Problem.*—Adjust and calculate an assigned triangulation system and plot the skeleton.

(c) *Methods.*—Observe the following program: (1) prepare forms for calculation and transcribe data; (2) adjust the angles of the triangulation system (see chapter on errors of surveying); (3) calculate the front and back azimuths of each line; (4) beginning with the base line compute the sides, to the nearest 0.001 foot; (5) calculate the latitudes and departures to the nearest 0.001 foot. (6) calculate the coordinates of the triangulation stations to the nearest 0.001 foot. In computing the coordinates of the stations take the mean of the values found by taking the different routes from the base line as the true value. (7) Plot the skeleton of the triangulation system to the prescribed scale by means of the coordinates of the points. The plotting sheet should be ruled off into squares very carefully before beginning the plotting. For this purpose use a steel straight edge and beam compass.

PROBLEM E12. SKETCHING TOPOGRAPHY.

(a) *Equipment.*—Small drawing board or plane table, plat of assigned field, 4H pencil.

(b) *Problem.*—Sketch in the roads, walks, buildings and five foot contours on the plat of the assigned field by eye having given the elevations of the ruling points.

(c) *Methods*—(1) Transfer from the level notes to the plat the elevations of the ruling points of the field. (2) Locate the roads, buildings, etc., on the map as nearly as possible in their relative positions (the topographers' estimate of distances should be frequently checked by pacing.) (3) Estimate the slopes and locate the contour points between the points of known elevation. (4) Join these points by smooth curved lines. (5) Finish the map in pencil, putting on a neat title, the scale of the map and a meridian. (6) Compare the finished map with a contour map furnished by the instructor.

PROBLEM E13. FILLING IN DETAILS WITH TRANSIT AND STADIA.

(a) *Equipment.*—Complete transit, 2 stadia rods, pocket tape.

(b) *Problem.*—Locate the topographic details of an assigned area with the transit and stadia.

Transitman J. Doe. Recorder R. Roe. Stadiamen {B. F. Keen. G. W. Sure

TOPOGRAPHIC SURVEY WITH TRANSIT AND STADIA

Shots	Stadia	Azim.	Vert.Ang	H. Corr.	Dist.	V. Corr.	Elev.	Object.	April 2, 1900.
At△16	Sight	on △18,	Azimuth 324°17'				205.21		(1 hour), Clear
1	4.20	290°00'	0°00'	0	421	0.0	205.2	Stubble.	and Warm.
2	4.58	181 24	-0 07	0	460	+0.9	206.1	"	Used Heller &
3	6.00	305 15	-0 01	0	601	+0.2	205.4	"	Brightly Trans-
4	7.85	309 30	-0 04	0	786	-0.9	204.3	E.&W. fence	it, Locker No. 6,
5	8.40	166 34	-2 10	1	842	+31.8	237.0	Pasture	and Stadia
6	5.65	328 10	-0 06	0	566	-1.0	204.2	E.&W. hedge	Boards 5 & 7.
7	8.40	148 45	-5 50	8	850	+850	290.2	Pasture.	Stadia ratio
8	3.31	328 30	-0 01	0	332	-0.1	205.1	Stubble.	k = 100.0
9	9.30	135 00	-10 14	29	860	+162.7	367.9	Pasture.	(adjustable
10	1.45	323 40	-0 10	0	146	-0.2	205.0	Stubble.	hairs)
11	8.40	245 00	-0 04	0	841	+1.0	206.2	Meadow.	f+c = 1.2
12	3.00	32 40	-0 02	0	301	+0.1	205.3	Stubble.	
13	7.24	242 22	-0 05	0	725	+1.0	206.2	N.&S. hedge.	(Use sketches
14	4.75	357 45	-0 09	0	476	-1.2	204.0	E.&W. "	whenever
15	4.10	140 10	-0 02	0	411	-0.2	205.0	Stubble.	necessary)
16	4.40	141 00	-0 15	0	441	-1.9	203.3	Slough.	
17	3.34	126 35	-0 03	0	335	+0.3	205.5	Stubble.	
18	5.55	355 30	-0 05	0	556	-0.8	204.4	Meadow.	
19	5.80	355 05	-0 24	0	581	-4.1	201.1	Bott. Ditch.	
20	4.10	75 25	-0 02	0	411	-0.2	205.0	Stubble.	
21	7.40	353 20	-0 03	0	741	-0.6	204.6	Fence.	
22	4.56	83 00	-0 05	0	457	-0.7	204.5	N.&S. hedge.	
23	6.82	16 30	-0 06	0	683	-1.2	204.0	Fence.	
				Note.	As a rule, the horizontal correction may be neglected.				

(c) *Methods*—(1) Set the transit over one corner of the field and set the A vernier to read the azimuth of a triangulation line. (2) Sight at the stadia rod held sidewise on the triangulation station at the other end of the given line, with the telescope normal and clamp the lower motion. (3) Read the intercept on the rod to the nearest 0.01 foot. Reduce this reading and check the ratio k by comparing the observed with the known distance. (4) Sight at target of the stadia rod or a rubber band at the height of the horizontal axis of the instrument above the first station and read the vertical angle to the nearest minute. (5) Unclamp the upper motion and take side shots to locate the topographic details. In taking the side shots read the intercept first, then set the middle cross-hair on the target and signal the rod man all right. The vertical angle and azimuth can then be read. Enough side shots should be taken to locate representative points, ridges, gullies, etc., on the surface that can be shown on the finished map. The scale of the map should therefore be known before beginning the field work. It is usually best to run along the top of a ridge and take side shots on both sides. (6) After all the side shots have been taken at the first triangulation station select a stadia station at a convenient point. (7) Sight at the edge of the stadia rod held on the stadia station and clamp the upper motion. (8) Read the A vernier which will be the azimuth of the course. (9) Read the intercept on the rod. (10) Measure the vertical angle as before. Set the transit over the stadia station and orient as at the first station. Take side shots as at the first station. Continue around the field, connecting the stadia stations by a closed traverse as in Problem E3. Record the field notes in the prescribed form. (11) Reduce the field notes by using either the slide rule, tables, or diagrams, and check in part by using one of the remaining methods. (12) Plot the stadia traverse and side shots using a protractor. Number each point plotted on the map and write its elevation just below the number in the form of a fraction. (13) Locate the contours by interpolating between the plotted points and complete the map in pencil on manila paper. (14) Trace the map if required.

PROBLEM E14. FILLING IN DETAILS WITH PLANE TABLE AND STADIA.

(a) *Equipment.*—Complete plane table (preferably with prismatic eyepiece), 2 stadia rods, engineers' divided scale, drawing paper, 6H pencil, pocket tape.

(b) *Problem.*—Locate the topographic details of an assigned area with the plane table and stadia.

(c) *Methods*—Follow the same methods as in Problem E13 except that the notes are to be plotted on the drawing paper in place of being recorded in the field book. Mark the point by number and write the elevation of each point under the number in the form of a fraction. Locate the contour points by interpolation on the map and connect the points by smooth curves. Complete the map in pencil and make a tracing if required.

PROBLEM E15. TOPOGRAPHIC SURVEY.

(a) *Equipment.*—Complete transit, 2 stadia rods, stakes, hubs, spring balance, pocket tape, stadia slide rule, seven-place logarithm table, (extra tripods, stadia reduction table stadia reduction diagrams, etc., as required).

(b) *Problem.*—Make a complete topographic survey of an assigned area and make a topographic map.

(c) *Methods*—(1) Make a reconnaissance and locate the triangulation stations. Care should be used to select the triangulation stations so that the sights will be clear and the triangles well formed. A system composed of quadrilaterals or more complicated figures will give more conditions and checks than a simple string of triangles. A system composed of simple triangles is sufficient for this survey. (2) Mark the triangulation stations with gas pipe monuments about 4 feet long, the exact point being marked by a hole drilled in a bolt screwed into a cap on the top of the gas pipe. (3) Measure the base line and base of verification as described in Problem E10. (4) Measure the angles by repetition as described in Problem D10. (5) Calculate the skeleton as described in Problem E11. (6) Establish permanent bench marks and determine their elevations and the elevation of the stations of the triangulation system by running duplicate levels with the engineers' level reading the rod to 0.001 foot. (7) Fill in the details with either the transit and stadia or the plane table and stadia, or both, as described in Problems E13 and E14. (8) Complete the map in pencil on manila paper, and after it has been approved by the instructor trace it on tracing linen. The title, meridian, scale, lettering and border should receive careful attention.

PROBLEM E16. LEVELS FOR PROFILE AND QUANTITIES FOR PAVING A STREET.

(a) *Equipment.*—Level, level rod, 4 flag poles, 100-foot steel tape, chaining pins, 50-foot metallic tape, hubs, axe.

(b) *Problem.*—Take level rod readings on the center line, right and left curb lines, right and left sidewalk lines, and right and left property lines to determine profiles and quantities for paving street. Plot profiles on Plate A profile paper to a scale of 40 feet to 1 inch horizontal and 4 feet to 1 inch vertical. Estimate the quantities of cut and fill and paving materials.

(c) *Methods*—(1) Locate the center line of the street and set flag poles on line about 400 feet apart by ranging in with the eye. (2) Drive a hub at one end of the street and call this point station zero. (3) Run a line of differential levels from the Standard B. M. to the zero end of the line. Read the rod to 0.01 foot. (4) Read the level rod to 0.1 foot on the ground at center hub. (5) Measure the distance out to the right curb line, right sidewalk and right property lines with the metallic tape and read the rod to 0.1 foot on the ground. (6) Repeat for the left side. (7) Chain along the center line to station 1. (8) Measure to the right and left from the chaining pin the required distances with the metallic tape and take rod readings as at station zero. (9) Repeat the process at each station and at abrupt changes intermediate. (10) Check the level circuit. (11) Make profile on Plate A paper, scales 40 feet to the inch horizontal and 4 feet vertical, indicating the several lines by conventional lines or colors. (12) Lay grade line as directed. (13) Show plat at bottom of profile. (14) Plot sections at scale of 20 feet to the inch and determine areas. (15) Compute quantities of earthwork, paving, etc. Follow form.

Leveler, J. Doe. Rodman, R. Roe. Chainmen { B. F. Keen. G. W. Swift.

LEVELS FOR PROFILE AND QUANTITIES FOR PAVEMENT ON WRIGHT ST.

Station	BS.	H.I.	E.S.	T.P.	B.M.	L. Prop. L40ft	L.S.Wlk. L37ft.	L.Gutter L20ft.	Centre	R.Gutter R20ft.	R.S.Wlk. R37ft	R.Prop. R43ft
+43	North Property Line, Healy St.					707.4/4.7	707.9/4.2	707.3/4.8	708.5/3.6	707.4/4.7	707.2/4.9	707.9/5.2
4+6	Centre Line, Healy St.					708.0/4.1	708.0/4.1	707.8/4.3	708.2/3.9	707.2/4.9	707.2/5.1	706.7/5.4
+76	South Property Line, Healy St.					707.5/4.6	707.5/4.6	707.6/4.5	708.6/3.5	707.1/5.0	706.7/5.4	706.6/5.5
3	North End of Bridge over Boneyard Creek					705.8/6.3	707.2/4.9	708.6/3.5	708.8/3.3	707.5/4.6	705.1/7.0	706.2/5.9
⊼	3.13	712.08										
●			7.89	708.35								
+62	South End of Bridge over Boneyard Creek					705.4/11.4	707.0/9.8	707.3/9.5	709.0/7.8	706.3/10.5	706.1/10.7	706.3/10.5
2						707.8/3.0	708.4/8.4	708.2/8.6	709.5/7.3	709.6/8.2	708.2/8.6	708.2/8.6
1						711.1/5.7	711.0/5.8	710.8/6.0	711.3/5.5	711.1/5.7	711.9/4.9	711.9/4.9
0	North Property Line, Green St.					713.9/2.9	714.0/2.8	713.6/3.2	714.0/2.8	713.9/2.9	714.9/1.9	714.2/2.6
⊼	0.88	716.84										
●			4.20	715.96								
⊼	1.07	720.16										
●			3.94	719.09								
⊼	3.03	723.03										
S.L.B.M.					720.00							

May 7, 1900 (3 hours) Warm and Windy.
Used Buff & Berger Dumpy Level, Locker No. 15.
Chained down centre of Street, lining in with transit poles, taking levels en route.

CHAPTER VII.

LAND SURVEYING.

Kinds of Surveys.—Surveys of land are of two kinds: (a) original surveys; (b) resurveys.

Original Surveys.—An original survey is made for the purpose of establishing monuments, corners, lines, boundaries, dividing land, etc. The survey of a townsite and the government survey of a section are examples of original surveys.

Resurveys.—A resurvey is made for the purpose of identifying and locating corners, monuments, lines and boundaries that have been previously established. The resurvey of a city block, or a survey to relocate a section corner are examples of resurveys.

Functions of a Surveyor.—In an original survey it is the function of the surveyor to make a perfect survey, establish permanent monuments and true markings, and make a correct record of his work in the form of field notes and a plat.

In a resurvey it is the function of the surveyor to find where the monuments, courses, lines and boundaries originally were, and not where they ought to have been. Failing in this it is his business to reestablish them as nearly as possible in the same place they were. No reestablished monument, no matter how carefully relocated will have the same weight as the original monument if the latter can be found. In making resurveys the surveyor has no official power to decide disputed points. He can only act as an expert witness. If the interested parties do not agree to accept his decision the question must be settled in the courts.

Rules for Resurveys.—The following rules may be safely observed in making resurveys.

(1) The descriptions of boundaries in a deed are to be taken as most strongly against the grantor.

(2) A deed is to be construed so as to make it effectual rather than void.

(3) The certain parts of a description are to prevail over the uncertain.

(4) A conveyance by metes and bounds will convey all the land included within.

(5) Monuments determine boundaries and transfer all the land included.

(6) When a survey and a map disagree the survey prevails.

(7) Marked lines and courses control courses and distances.

(8) The usual order of calls in a deed is: natural objects, artificial objects, course, distance, quantity.

(9) A long established fence line is better evidence of actual boundaries than any survey made after the monuments of the original survey have disappeared.

(10) A resurvey made after the monuments have disappeared is to determine where they were and not where they ought to have been.

(11) All distances measured between known monuments are to be pro rata or proportional distances.

If the above rules do not cover the case in question special court decisions on that particular point should be consulted.

THE UNITED STATES RECTANGULAR SYSTEM OF PUBLIC LAND SURVEYS.

Historical.—The United States rectangular system of subdividing lands was adopted by congress May 20, 1785. The first public land surveys were made in the eastern part of the present state of Ohio under the direction of Capt. Thomas Hutchins,* Geographer of the United States, and were known as the "Seven Ranges". The townships were six miles square, and were laid out in ranges extending northward from the Ohio river; the townships were numbered from south to north, the ranges from east to west. In these initial surveys only the exterior lines of the town-

*The earliest published reference to the rectangular system of land surveys is found in an appendix to "Bouquet's March," published in Philadelphia, 1764. Hutchins was engineer with this expedition to the forks of the Muskingum river, and wrote the appendix. (See reprint by Robt. Clarke, Cincinnati.)

ships were run, but mile corners were established on the township lines, and sections one mile square were marked on the plat and numbered from 1 to 36, commencing with section 1 in the southeast corner and running from south to north in each tier to 36 in the northwest section.

The act of congress approved May 18, 1796, provided for the appointment of a surveyor general and changed the law relating to the surveys of public lands. Under this law the townships were subdivided into sections by running parallel lines two miles apart each way and setting a corner at the end of each mile. This law also provided that the sections be numbered beginning with section 1 in the northeast corner of the township, thence west and east alternately to 36 in the southeast section. This is the method of numbering still in use, shown in Figs. 33 and 34.

The act of congress approved May 10, 1800, required that townships be subdivided by running parallel lines through the same from east to west and from south to north at a distance of one mile from each other. Section corners and half section corners on the lines running from east to west were required to be set. The excess or deficiency was to be thrown into the north and west tiers of sections in the townships.

The act of congress approved February 11, 1805, required that interior section lines be run every mile; that corners be established every half mile on both townships and section lines; that discrepancies be thrown on the north and west sides of the township. This act of congress further provided "that all corners marked in the original surveys shall be established as the proper corners of sections, or subdivisions of sections; and that corners of half and quarter sections not marked shall be placed as nearly as possible "equidistant" from those two corners which stand on the same line. The boundary lines actually run and marked shall be established as the proper boundary lines of the sections or subdivisions for which they were intended; and the length of such lines as returned by the surveyor shall be held and considered as the true length thereof, and the boundary lines which shall not have been actually run and marked as aforesaid shall be ascertained by running straight lines from the established corners to the opposite corresponding corners." Under this law, which is still the

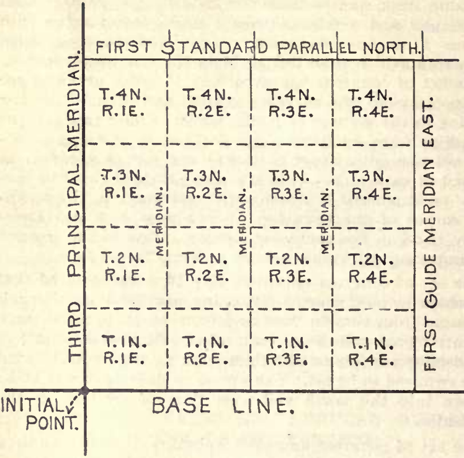


Fig. 32.

established rule of procedure, each reported distance between established monuments is an independent unit of measure.

The revised instructions issued in 1855 required that the sections be subdivided as shown in Fig. 33, the full lines, representing "true" lines, are parallel to the east exterior line of the township, and the dotted lines, representing "random" lines, close on corners previously established. The order of the survey of the interior section lines is indicated by the small numerals. Double corners on the north and west township lines, which were common in the earlier surveys, were thus avoided in the revised practice.

Laws Inconsistent.—It is obviously impossible to preserve a true rectangular system on a spherical surface, owing to the convergency of meridians.* To harmonize the methods of making surveys, the General Land Office has issued instructions for the survey of public lands from time to time.

DETAILS OF SURVEY.—The details of the survey are taken up in the following order: (1) selection of initial points; (2) establishment of the base line; (3) establishment of the principal meridian; (4) running standard parallels; (5) running the guide meridians; (6) running the township exteriors; (7) subdividing the township; (8) meandering lakes, rivers, streams, etc. See Figs. 32 and 33.

Initial Points.—Initial points from which to start the survey are established whenever necessary under special instructions prescribed by the Commissioner of the General Land Office.

Base Line.—The base line is extended east and west from the initial point on a parallel of latitude. The proper township, section and quarter corners are established and meander corners at the intersection of the line with all meanderable streams, lakes, or bayous. Two sets of chainmen are employed and the mean of the two measurements is taken as the true value. When the transit is used, the base line—which is a small circle parallel to the equator—is run by making offsets from a tangent or secant line, the direction of the line being frequently checked by an observation of Polaris.

Principal Meridian.—The principal meridian is extended either north or south, or in both directions from the initial point on a true meridian. The same precautions are observed as in the measurement of the base line.

Standard Parallels.—Standard parallels, which are also called correction lines, are extended east and west from the principal meridian, at intervals of 24 miles north and south of the base line. They are surveyed like the base line.

Guide Meridians.—Guide meridians are extended north

---

*The angular convergency, a, of two meridians is m sin L, where m is the angular difference of longitude of meridians and L is the mean latitude of the two positions. The linear convergency, c, for a length, t, is t sin a. For latitude 40°, the difference between the north and south sides of a township is 0.60 chains.

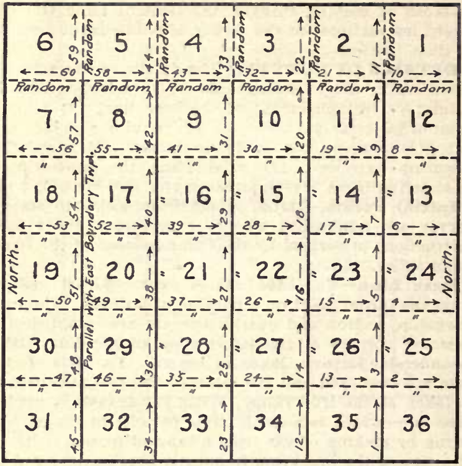
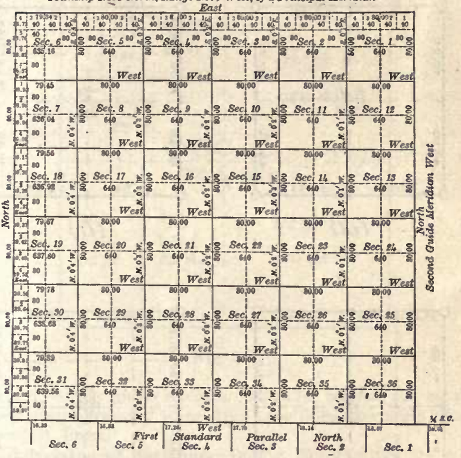


Fig. 33.

from the base line, and standard parallels, at intervals of 24 miles east and west from the principal meridian, in the manner prescribed for running the principal meridian. When existing conditions require that guide meridians shall be run south from the base or correction lines, they are initiated at properly established closing corners on such lines.

Township Exteriors.—The township exteriors in a tract 24 miles square, bounded by standard lines, are surveyed successively through the block, beginning with the south-western township. The meridional boundaries are run first form south to north on true meridians with permanent corners at lawful distances; the latitudinal boundaries are run from east to west on random or trial lines and corrected

Township No. 5 North, Range No. 9 West, of a Principal Meridian



The above plot represents a theoretical township with perfect subdivisions, contiguous to the north side of a Standard Parallel; in assumed Latitude 42 15′ N., and Longitude 100°00′ W. of Gr. Area 23024.16 A.

Fig. 34.

back on true lines. Allowance for the convergency of meridians is made whenever necessary.

Township Subdivision.—A true meridian is established at the southeast corner of the township and the east and south boundaries of section 36 are retraced. Then beginning at the corner to sections 35 and 36 on the southern boundary, a line is run north parallel to the range line, corners are established at a distance of 40 and 80 chains; from the last named corner a random line is run eastward, parallel to the south boundary line of section 36, to its intersection with the east boundary of the township. A temporary corner is set at a distance of 40 chains, and a permanent corner is afterwards established midway be-

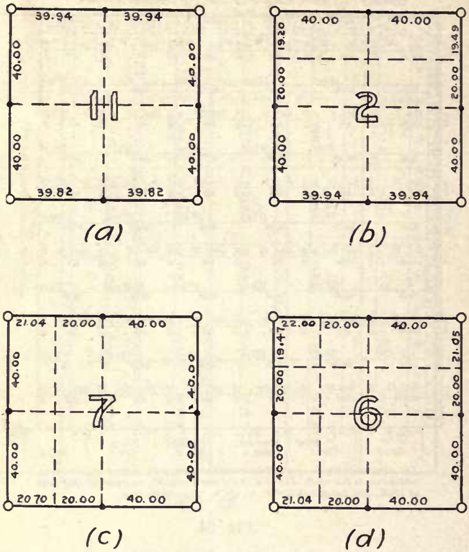


Fig. 35.

tween the two permanent corners. The other corners are located in a similar manner, as shown in Fig. 33. The lines closing on the north and west boundary lines of the township are made to close on the section corners already established. A theoretical township with perfect subdivisions is shown in Fig. 34.

Meandering.—Navigable rivers and other streams having a width of three chains and upwards are meandered on both banks, at the ordinary high water line by taking the general courses and distances of their sinuosities. The meanders of all lakes, navigable bayous, and deep ponds of the area of twenty-five acres and upwards are surveyed as

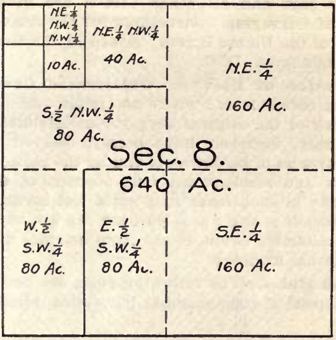


Fig. 36.

directed for navigable streams. Meander corners are established where meander lines cross base lines, township lines, or section lines.

Subdivision of Sections.—In Fig. 35, (a) gives the subdivision of an interior section, (b) of section 2 on the north side, (c) of section 7 in the west tier, and (d) of section 6 in the northwest corner.

Description of Land.—Land is described in the rectangular system by giving its location in a civil township; for example, in Fig. 36, the northwest quarter, containing 160 acres, would be described as: N E ¼, Sec. 8, T 19 N, R 9 E, 3 P. M. The ten acre lot indicated in the northwest quarter would be described as: S E ¼, N W ¼, N W ¼, Sec. 8, T 19 N, R 9 E, 3 P. M.

Corners.—The corner monuments may be as follows: (a) stone with pits and earthen mound; (b) stone with mound of stone; (c) stone with bearing trees; (e) post in mound of earth; (f) post in mound of stone; (g) post with bearing trees; (h) simple mound of earth or stone; (i) tree without bearing trees; (j) tree with bearing trees; (k) rock in place, etc. The trees on line are required to be blazed. The size, markings and proper corners to be used in any particular case and all other details are given in the "Manual of Surveying Instructions for the Survey of Public Lands of the United States," issued by the General Land Office, Washington, D. C.

Restoration of Lost or Obliterated Corners.*—An obliterated corner is one where no visible evidence remains of the work of the original surveyor in establishing it. Its location may, however, have been preserved beyond all question by acts of landowners, and by the memory of those who knew and recollect the true position of the original monument. In such cases it is not a lost corner.

A lost corner is one whose position can not be determined beyond reasonable doubt, either from original marks or reliable external evidence.

General Rules.—The following rules are derived from a brief synopsis of congressional legislation relating to surveys.

(1) The boundaries of the public lands established and returned by the duly apponted government surveyors, when approved by the surveyors general and accepted by the government, are unchangeable.

(2) The original township, section, and quarter-section corners established by the government surveyors must stand as the true corners which they were intended to represent, whether the corners be in place or not.

(3) Quarter-quarter corners not established by the government surveyors shall be placed on the straight lines joining the section and quarter-section corners and midway between them, except on the last half mile of section lines closing on the north and west boundaries of the townships, or on other lines between fractional sections.

(4) All subdivisional lines of a section running between corners established in the original survey of a township must be straight lines, running from the proper corner in one section line to its corresponding corner in the opposite section line.

(5) That in a fractional section where no opposite corresponding corner has been or can be established, any re-

*Circular on the "Restoration of Lost and Obliterated Corners and Subdivision of Sections," General Land Office, Washington, D. C.

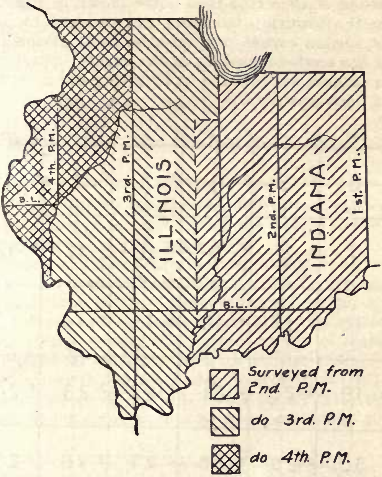


Fig. 37.

quired subdivision line of such section must be run from the proper original corner in the boundary line due east and west, or north and south, as the case may be, to the water course, Indian reservation, or other boundary of such section, with due parallelism to section lines.

Locations of Principal Meridians.—Principal meridians have been established as the needs of the surveys warranted. The surveys in the state of Indiana were made with reference to the 2nd Principal Meridian, and those of Illinois with reference to the 2nd, 3rd and 4th Principal Meridians. See Fig. 37. There are twenty-four principal meridians in all, the locations of which are given in the "Manual of Instructions," mentioned above.

Abridging Field Notes.—The government surveyors use the method of abridging field notes shown in Fig. 38. Corners in the township boundary are referred to by letter; interior section corners are referred to by giving the numbers of the sections meeting at the corner; interior quarter section corners are referred to by giving the number on the section lines produced.

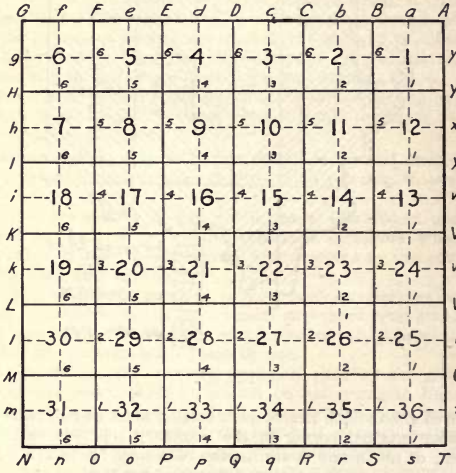


Fig. 38.

SURVEYS BY METES AND BOUNDS.

That portion of the United States settled before the adoption of the rectangular system was surveyed by the method of metes and bounds. For the most part these surveys were very irregular and often involved complex and conflicting conditions. The entire eastern portion of the United States, and the state of Kentucky, were surveyed in this manner, and further examples are found in the French surveys in the states of Michigan, Indiana, Illinois, Missouri, Louisiana, etc., and the Spanish surveys of Texas, California, etc. The general principles underlying the questions of ownership, priority of survey, the restoration of lost corners, etc., are identical whatever the system of survey used.

PROBLEMS IN LAND SURVEYING.

PROBLEM F1. INVESTIGATION OF A LAND CORNER.

(a) *Equipment.*—Digging outfit, tape, etc, as required.

(b) *Problem.*—Collect complete evidence relative to an assigned land corner, and after giving due weight to the same, make a decision as to the true corner.

(c) *Methods.*—(1) Make careful examination of the official field notes and records pertaining to the land corner in question and make extracts from the same for further reference. (2) Seek oral evidence from those acquainted with the history of the corner. (3) Make a survey of fence lines and other physical evidence, such as witness trees or their stumps, etc., near the corner under investigation. (4) Make careful examination of the site of the corner with the digging outfit; the digging should be done cautiously so as to

J. Doe,
Surveyor.

INVESTIGATION OF S.W. CORNER SECTION 8, T. 19 N., R. 9 E., 3D P.M.

Original United States Field Notes, on file at Court House at Urbana, Ill., describe the S.W. Cor., Sec. 8, T. 19 N., R. 9 E., 3 P.M., as "Post in Mound", the corner being located on the prairie remote from the heavy timber which surrounds the other three corners of the section. Original Survey was made about 1822.

On Oct 25, 1896, Col. S. T. Busey, when asked for information about the corner under investigation, stated that about 1850, when he was a boy, Mr. Campbell, who was then County Surveyor, was called on to re-establish the S.W. Cor. Sec. 8. At the time mentioned the section lines near the corner were occupied by rail fence. Col. Busey says that his father (a pioneer settler) pointed out to the surveyor near the fence corner evidences of a mound which he believed marked the original U.S. Survey corner. Mr. Campbell, the surveyor, dug carefully at the spot and found the decayed point of a sassafras stake which unquestionably marked the true position of the "Post in Mound" established some 25 years or more previous to Campbell's resurvey. Col. Busey states that he himself carried the boulder which was set in place by the County Surveyor to perpetuate the section corner, and that this monument was not disturbed until it was replaced by a much larger stone when the roads were opened up on the section lines.

This stone stood 18" or so above the level of the road for many years. About 1894 it was carefully lowered by the Street Commissioner under the direction of the City Engineer of Urbana. Resurveys made since the stone was lowered, indicate that its present position is identical with that previous to the change.

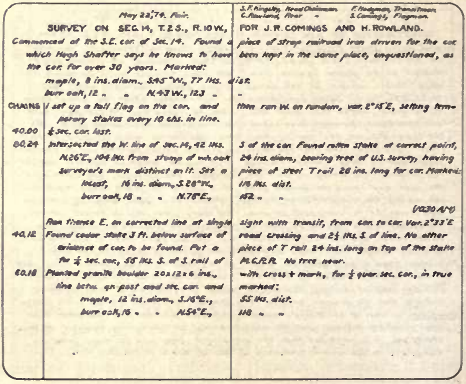
Conclusion. In view of Col. Busey's valuable statement with the corroboration from other credible sources, and the entire absence of conflicting evidence of any character, it is concluded that the monument now and for many years so recognized is the true S.W corner of Sec 18, T. 19 N. R. 9 E., 3D. P.M avoid disturbance of existing stakes or other monuments. (5) If more than one monument be found, make due record of their character and positions, and make further inquiry respecting them. (6) If no monument of any sort be found at first, continue the search diligently and do not give up finding the true corner as long as there is a remote chance of locating it. In any event, avoid wanton disturbance of any object or evidence that may have a bearing on the same. Keep clear and concise record.

PROBLEM F2. PERPETUATION OF A LAND CORNER.

(a) *Equipment.*—Digging outfit, a large boulder or other permanent monument, cold chisel, hatchet, plumb bob, string, stakes.

(b) *Problem.*—Replace a temporary land corner by a permanent monument.

(c) *Methods.*—(1) Uncover the identified temporary monument and carefully determine the true point with consistent exactness. (2) Reference out the point by driving two pairs of stakes with strings stretched so as intersect squarely over the corner. (3) After carefully checking the



May 22,'74. Fair. S. F. Kingsley, Head Chainman; C. Rowland, Rear "; F. Hodgman, Transitman; S. Comings, Flagman.

SURVEY ON SEC. 14, T. 2 S., R. 10 W., FOR J. R. COMINGS AND H. ROWLAND.

Commenced at the S.E. cor. of Sec. 14. Found a piece of strap railroad iron driven for the cor. which Hugh Shafter says he knows to have been kept in the same place, unquestioned, as the cor. for over 30 years. Marked:
maple, 8 ins. diam., S.45°W., 77 lks. dist.
burr oak, 12 " " N.43 W., 123 " "

CHAINS	
	I set up a tall flag on the cor. and then ran W. on random, var. 2°15'E, setting temporary stakes every 10 chs. in line.
40.00	¼ sec. cor. lost.
80.24	Intersected the W. line of Sec. 14, 42 lks. S of the cor. Found rotten stake at correct point, N.26°E., 104 lks. from stump of wh. oak 24 ins. diam., bearing tree of U.S. survey, having surveyor's mark distinct on it. Set a piece of steel T rail 28 ins. long for cor. Marked: locust, 16 ins. diam., S.28°W., 116 lks. dist. burr oak, 18 " " N.78°E., 152 " " (10:30 A.M.)
	Run thence E. on corrected line at single sight with transit, from cor. to cor. Var. 2°33'E
40.12	Found cedar stake 3 ft. below surface of road crossing and 2½ lks. S. of line. No other evidence of cor. to be found. Put a piece of T rail 24 ins. long on top of the stake for ¼ sec. cor., 55 lks. S. of S. rail of M.C.R.R. No tree near.
60.18	Planted granite boulder 20x12x6 ins., with cross + mark, for ⅛ quar. sec. cor., in true line betw. qr. post and sec. cor. and marked: maple, 12 ins. diam., S.16°E., 55 lks. dist. burr oak, 16 " " N.54°E., 118 " "

referencing, dig out the old monument to a depth sufficient to receive the boulder and permit its top to set several inches beneath the natural surface if located in a road or where disturbance is probable. (4) Cut a plain cross mark on the top of the stone, and set it in place in the hole, packing the earth about it, and testing the position of the mark by means of the reference stakes and strings and plumb bob; finally leave the boulder set firmly in the correct position. (5) Make reference measurements to suitable permanent points such as marks on curbing, gas pipes, witness trees, etc., selected with respect to good intersections, and make reliable record of the witness notes after checking the same. (Other forms of permanent monuments are: gas pipe; fish plate; section of T-rail; farm tile or vitrified pipe filled with cement mortar; post hole filled with mortar; special solid monument burned like farm tile; special casting similar to a gas main valve box, with hole in top to receive flag pole; etc.)

PROBLEM F3. REESTABLISHING A QUARTER-SECTION CORNER.

(a) *Equipment.*—Transit party outfit, digging tools, etc.

(b) *Problem.*—Reestablish a quarter-section corner that has been obliterated or lost.

(c) *Methods.*—(1) Collect and record all the available evidence which may assist in the discovery and identification of the corner. Examine the field notes of the original survey, the surveyors' plat book and the county atlas on file at the court house, and make diligent inquiry for credible and competent information, either written or oral as to the location of the corner. (2) Make a careful search for the monument. Trace all the lines of the original survey, paying particular attention to bearing and sight trees. Dig in all the places indicated by the different lines and give up the search only after you have exhausted every possible clue. (3) If the corner cannot be found, reestablish it, giving due weight to all the evidence. The surveyor should remember that the corner should be reestablished where it originally was and not where it ought to be. After having located a stake at the supposed location of the original monument ,reference it out and renew the search. (4)

After the monument has been relocated, mark it in a permanent manner as indicated in Problem F2, by a stone with a cross cut in its top or with a gas pipe well driven into the ground. Reference it out to at least two permanent objects selected with a view to securing a first class intersection. Make a careful record and preserve consistent accuracy in the work.

PROBLEM F4. REESTABLISHING A SECTION CORNER.

(a) *Equipment.*—Transit party outfit, digging tools, etc.

(b) *Problem.*—Reestablish an obliterated or lost section corner.

(c) *Methods.*—Follow the various methods described in Problem F3, giving special attention to the search for the original corner; upon failing to find trace of it, run out lines with reference to the section, quarter, and quarter-quarter corners in the four directions, with linear measurements from the same and finally reach the most consistent decision with reference to such survey lines, ownership lines, fences, hedges, road centers, etc. (A fruitful cause of disturbance of section and other corners is careless use of road graders, or the failure to lower the corner sufficiently below the surface of the road.)

PROBLEM F5. RESURVEY OF A SECTION.

(a) *Equipment.*—Transit party outfit, digging tools, etc.

(b) *Problem.*—Make a resurvey of an assigned section.

(c) *Methods.*—(1) Make extracts from the field notes of the original survey and of all resurveys on file at the court house, and other notes that may be of value. Make diligent inquiry among the property owners for evidence as to the location of corners. (2) Retrace the lines, recording the location of old fences, timber markings and other evidences as to prior recognition of lines and corners. Use consistent accuracy. Record the original notes as given in the forms. Record the field notes in narrative style using the designation of corners as given in the resurvey plat in the form. Make a plat of the section in the manner prescribed by state law for a resurvey.

Surveyor J. Doe

INVESTIGATION OF LAND CORNERS OF SEC. 8, T 19 N, R 9 E, 3RD P.M.

COLLECTION OF EVIDENCE. April 25, 1899.

Extracts from Surveyors Plat Book of Resurveys of Champaign County.

Nov 5, 1897, found in the County Recorder's office at Urbana, Ill., the "Surveyor's Plat Book" containing plats of townships showing existing monuments and subdivisions of sections made by the County Surveyor, with certificates of various resurveys. Made the following extracts relating to Sec 8, T 19 N, R 9 E, 3RD P.M.:—

(From p. 156)

"Dec 3, 1876, Surveyed at the request of F. Adams the east line of Sec. 6. Beginning at a stone previously planted at N.E. Cor of said section, and running thence S. to S.E. Cor of same where I found a stone previously set by John Thrasher and Lewis Sommers, divided the distance pro rata and set Cor at N.E. Cor of S.E. ¼ of same."

(Signed) Thos B Kyle.
Co Surveyor

(From p. 157)

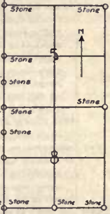
"Apr 11, 1884 Surveyed by request of S.T. Busey the W lines of Secs 8 and 5. Beginning survey at S.W. Cor. Sec 8 where a stone is planted and running thence N. to N.W. Cor Sec. 5, found an excess of 40 lks, corrected back same on to a stone planted by Lewis Sommers at ¼ Sec Cor on line between Sec 5 and 6.

I also planted a stone at Sec. Cor (5-6-7-8) and made the following witnesses to the corner viz: A double Burr Oak 15" diam. bearing N 60½° E 101½ lks, also a W. Oak 14" diam. bearing N 35½° E 188 lks

"I also set a stone at the N W cor. of the S.W ¼ of the S.W ¼ of Sec. 5"

(Signed) Thos B Kyle
Co Surveyor

(Portion of Plat on p. 155, showing existing monuments)



Surveyor J Doe

INVESTIGATION OF LAND CORNERS OF SEC 8, T 19 N, R 9 E, 3RD P.M.

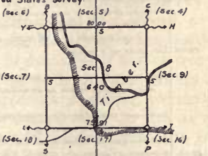
COLLECTION OF EVIDENCE (Continued) April 25, 1899.

Extracts from Field Notes of Original United States Survey.

Nov 4 1897, found in the County Treasurer's Office at Urbana, Ill., the Plat Book containing Plats and Abstracts of Field Notes of Original United States Survey of Champaign County, and made the following extracts relating to Sec 8, T 19 N, R 9 E, 3RD P.M.:—

DESCRIPTIONS OF ORIGINAL CORNERS. (P. 50)

CORNERS. Designation	WITNESS TREES Kind	Inches Diameter	Courses They Bear	Links Distant
Sec Cors.				
4, 5, 8, 9	Ash B Oak	24 14	S 58 E N 62 W	35 26
5, 6, 7, 8	W Oak W Oak	28 20	N 88 E N 36 E	230 272
7, 8, 17, 18,	Post in mound			
8, 9, 16, 17,	B Walnut B Walnut	24 24	N 32 E S 10 W	44 42
¼ Sec Cor				
N to Y of 5	Elm Elm.	12 8	S 67 W N 78 E	20 30
I - X - 5	W. Oak W Oak	6 6	N 58 E S 26 E	23 20
R - C - 5	Ash Elm	22 8	S 7 E N 30 E	28 13
S - B - 5	Post in mound			



DESCRIPTIONS OF "OBJECTS ON THE LINES." (P. 75)

DESIGNATION	DISTANCES Chs. Lks.	DESCRIPTION
N. betw. 8 & 9	25 00	Brook leading N. thence along the channel of the same 15 chs then leaving it running Ely
	50.19	Ash 12 in. diam.
E - 8 - 17	24 50	Brook 5 lks rs N. Ely.
	39 00	Entered timber bs N & S.
E - 5 - 8	4 00	Entered timber bs N & S.
	16.50	Brook 60 lks rs Sly.

RESURVEY OF SEC.27, T.12 N., R.16 W., 3RD. P.M.

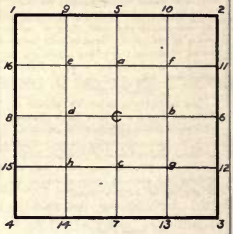
CHAINS	
	Began at 1. Found stake in place and both bearing trees standing. Planted stone 25"x8"x6"; marked + for cor.
	Thence N. on random, var. 2°30' E., setting temp. stakes every 10 chs.
80.22	Intersected sec. line 26 lks. W. of 5.
	At 5 found rotten stake at correct point, S.28°W, 66 lks. from stump of wh. oak, bearing tree of U.S. survey. Drove stake for cor. and put broken earthenware and glass around it. Marked wh. oak, 12" diam, N 66°E, 42 lks., also wh. oak 18" diam, N 34°W, 63 lks.
	From 5 ran E. on random, setting temp. stakes every 10 chs.
39.92	Intersected sec. line 12 lks. N. of 2.
	At 2 found earthen post in correct position and bearing trees of resurvey standing.
	Thence W. on corrected line.
9.98	Set stake on true line.
(cont'd	on next page)

S.H. Smith, Head Chainman. L.B. Brown, Axman.
J.E. Wilson, Rear " G.W. Smith, Flagman.

FOR THE ESTATE OF JOHN W. SMITH.

July 12, '82. Cloudy with showers.

RESURVEY REFERENCE PLAT.



RESURVEY SEC. 27, SMITH ESTATE (CONTINUED).

CHAINS.	
19.98	(Line 5-2 cont'd) At 10 set stake with stones around it and marked: 'pine, 12" diam, N.46°W., 79 lks. red oak, 24" " S.19½ W., 72 "
29.94	Set stake on true line.
	From 10 ran S. on random, var. 2°18' E., and set temp. stakes at 20 and 40 chs.
	Then went to 6. Found post and bearing trees of resurvey standing.
	Ran thence W. on random, var. 2°20' E.
20.02	Intersected random line from N. 6 lks. S. of temp. stake.
40.18	Intersected random ¼ line 8 lks. N. of temp stake.
80.04	Intersected sec. line 10 lks. S. of 8.
	Cor. post dug out in road. Set iron plow beam for cor. S.28°W., 76 lks. from bearing tree of U.S. survey.
	Thence E. on corrected line.
39.99	At intersection of quarter lines set post.

PROBLEM F6. RESURVEY OF A CITY BLOCK.

(a) *Equipment.*—Transit, 100-foot steel tape, chaining pins, axe, hubs, stakes, 4 pieces one-inch gas pipe 2 feet long, notes of previous surveys, etc.

(b) *Problem.*—Make a resurvey of an assigned city block.

(c) *Methods.*—(1) Procure full notes of all the surveys and resurveys of the assigned block from the records at the court house and from any other source available. (2) Make a resurvey of the block, using the notes, and drive hubs for temporary corners. (3) Compute the latitudes and departures of the courses, and if consistent, balance the survey. (4) If the corners of the block as located are consistent with the existing property and street lines, drive gas pipes as permanent corners. (5) Subdivide the block into lots as shown in the notes. (6) Make a plat of the block on manila paper to the prescribed scale, showing block and lot lines, distances and angles obtained in making the survey, the names of the owners of the property and the names of the streets. Prepare a surveyors' certificate as provided by law. Trace the map if required. (The accuracy attained should be based on the valuation and other local conditions. Before beginning the survey use every possible care to find the corners with reference to which the original survey was made. When lots are sold by number, the excess or deficiency should be divided pro rata. However, when lot lines have been long acquiesced in, it is doubtful if the courts will uphold the surveyor in interfering with the ancient lines of ownership. It then becomes necessary either to make a compromise survey that will be satisfactory to the owners, or to make a survey that is strictly according to the letter of the law, and submit the map and certificate to the courts for settlement. The surveyor should remember that he is simply an expert witness and that he has no final judicial powers.)

PROBLEM F7. RESURVEY BY METES AND BOUNDS.

(a) *Equipment.*—Transit party outfit, digging tools, etc.

(b) *Problem.*—Make a resurvey of an assigned tract whose original survey was made by metes and bounds.

(c) *Methods.*—(1) Collect full notes and data relating to the monuments, magnetic bearings, magnetic variation, date of survey, lengths of lines, etc. (2) Make a careful investigation of the lines and corners on the ground and make notes of any evidence there found. (3) Locate and identify with certainty as many as possible of the original monuments; where double or contested corners exist, locate each definitely for further reference; if corners are generally lacking or doubtful, concentrate attention on at least two which give most promise of definite relocation, and reestablish these corners as carefully as possible. (4) Having at least two corners, retrace by random line the perimeter of the tract according to the original description, beginning at one and closing on the other corner; set temporary corner stakes at the several points; note the linear and angular error of closure of the random traverse on the last monument. (5) Calculate the latitudes and departures of the random survey, and determine the angular and linear relations between the random and the original survey; also fix the position of the several random stakes relative to the supposed true positions of the respective corners. (6) Set stakes in the true positions, as calculated, reference them out, and renew the search for the original monuments. (7) Finally reestablish each corner in the most consistent position, put permanent corners in place, and take witness notes for each, making complete notes of the proceedings. Follow form.

PROBLEM F8. PARTITION OF LAND.

(a) *Equipment.*—Transit party and digging outfits, etc.

(b) *Problem.*—Make a partition of an assigned tract of land in accordance with instructions.

(c) *Methods.*—(1) Make the necessary resurveys of the assigned tract, identifying original monuments, and reestablishing lost corners as required. (2) Make a plat of the partition. (3) Subdivide the land and set permanent corners; carefully establish witnesses to the corners and secure witness notes. (4) Prepare and file plat and description as required by law.

J. Doe, Surveyor. Mar. 10, 1892.

RESURVEY OF "MISSION RIDGE" PUBLIC ROAD FOR J. D. CLARK.

Consulted County Records and confirmed following meander notes for center line of highway as described in J. W. Martin's deed to J. D. Clark. "N. 62° E., 14 ch.; N 43½° E., 8 ch.; N. 5° W., 12 ch.; N. 72½° E., 10.25 ch.; S. 12° W., 6.43 ch." Description referred to stones at beginning and ending points.

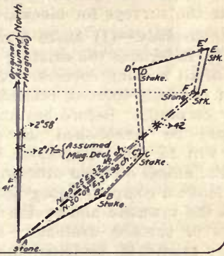
Found first stone projecting above road, but could not locate last corner.

Began at first monument and ran on random according to meander notes, with 2° 17' E. as magnetic declination. Drove temporary stake at each deflection point and made careful search for monuments. Found no corners at intermediate points, but identified marked boulder as true corner at closing point 62 links due west of last stake of random.

Made careful calculation of notes for shifting over from random to true corners. (See plat opposite and calculations on next pair of pages.)

Transferred corners according to calculations and renewed search for original monuments, keeping close watch for decayed stakes, but without success.

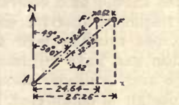
Set stone at each true corner.



CALCULATIONS, RESURVEY OF "MISSION RIDGE" ROAD.

Data transcribed from pp. Copy O.K.

Sta. (Random Line)	Bearing	Dist. Ch.	Lat. Ch.	Dep. Ch.	Tot. Lat. Ch. (N)	Tot. Dep. Ch. (E)
A	N 62° 00' E	14.00	N 6.57	E 12.36		
B	N 43° 30' E	8.00	N 5.80	E 5.57	6.57	12.36
C	N 5° 00' W	12.00	N 11.95	W 1.05	12.37	17.87
D	N 72° 30' E	10.25	N 3.08	E 9.78	24.32	16.82
E	S 12° 00' W	6.43	S 6.29	W 1.34	27.40	26.60
F			N 27.40	E 27.65	21.11	25.26
			S 6.29	W 2.39		
			N 21.11	E 25.26		
(Original Survey in Terms of Resurvey.)						
A'	N 61° 18' E	13.80	N 6.63	E 12.10		
B'	N 42° 48' E	7.88	N 5.78	E 5.35	6.63	12.10
C'	N 5° 42' W	11.83	N 11.77	W 1.17	12.41	17.45
D'	N 71° 48' E	10.10	N 3.15	E 9.60	24.18	16.28
E'	S 11° 18' W	6.34	S 6.22	W 1.24	27.33	25.88
F'			N 27.33	E 27.05	21.11	24.64
			S 6.22	W 2.41		
			N 21.11	E 24.64		



Notes for Shifting from Random to True Corners.

Line	Difference. Lat. Lks.	Difference. Dep. Lks.	Dist. Lks.	Bearing.
BB'	N 6	W 26	26.7	N 77° 00' W
CC'	N 4	W 42	42.2	N 84° 34' W
DD'	S 14	W 54	55.8	S 75° 28' W
EE'	S 7	W 72	72.3	S 84° 27' W
FF'	0	W 62	62.0	W

Note. *The above solution is based on the assumption that the error of closure of 0.62 ch. at FF' is due to difference of both chain and needle. Distances AF and AF' and angles NAF and NAF' were calculate, giving chain and needle corrections.*

PROBLEM F9. DESIGN AND SURVEY OF A TOWN SITE (OR ADDITION.)

(a) *Equipment.*—Equipment for topographic survey for both field and office.

(b) *Problem.*—Make a preliminary topographic survey of the proposed town site (or addition) design the plat, and make the surveys for blocks, lots, etc.

(c) *Methods.*—(1) Make a careful resurvey of the entire tract. Reference the existing monuments and carefully relocate all missing corners. (2) After the monuments have been carefully located, remeasure the distances and angles very carefully. Before beginning the chaining, a standard should be established as described in Problem A23. (3) Fill in the topographic details with the transit and stadia, unless directed otherwise, using consistent accuracy. (4) Make a complete topographic map of the tract. (5) Design the townsite and sketch it in on the map. The questions of surface drainage, sewerage, possible overflow, street gradients, principal thoroughfares, diagonal streets, alleys, etc., should be carefully considered. The streets should be of ample width and laid out with reference to ease of grading both the street and the adjacent property. Residences should face desirable streets and the cross streets in the residence district should not be too numerous. The principal thoroughfare should pass through the business portion and have minimum gradients. The system of sewerage and drainage should be worked out roughly before the design is completed. Much expensive construction can be avoided by using care in designing the town site. (6) Make preliminary profiles of all the streets on Plate A profile paper to the prescribed scale. (7) Carefully locate the block and other important corners and mark them by permanent monuments of stone, gas pipe, tiling, etc. (8) Subdivide the blocks into lots and mark the lot corners by means of gas pipes or hubs. (9) After the streets have been located carefully, take levels on the same, make profiles, and lay grade lines for all streets, sidewalks, and improvements.

Use accuracy consistent with the value of the property throughout the problem. Make a careful record of the notes. Complete the maps and profiles.

CHAPTER VIII,

RAILROAD SURVEYING,

Classification —For the purpose of class instruction, railroad surveying will be discussed under the following heads: (1) curve practice, (2) reconnaissance, (3) preliminary survey, (4) location survey, (5) construction, (6) maintenance.

Curve practice is designed to give the student familiarity with the methods of running curves so that the location survey may be made without needless delay. It consists of a series of typical problems covering the usual range of conditions found in such surveys.

The reconnaissance is a rapid preliminary examination of a district or area for the purpose of selecting ruling points to control the general routes of the preliminary survey lines. The distances are paced or scaled from a map; elevations are determined by means of the barometer or hand level.

The preliminary survey is designed to obtain information and to obtain it rapidly, as a guide in making the location survey. A rapid deflection angle traverse is run, following the general route of the proposed line, but keeping in clear ground as far as may be to gain time; levels are run, topography including contours taken, the map made, and one or more location lines projected on the map.

The location survey fixes the exact lines, including the curves, preparatory to building the proposed railroad. Some engineers prefer to run one or more trial location lines, but it is best practice to locate the line as projected on a reliable contour map.

Construction surveys are made for the purpose of fixing the roadbed limits and other constructive details, and estimating earthwork and other quantities.

Maintenance surveys and resurveys are made after the line is built, for ballasting, yard construction or other purpose.

Field Organization of Class.—In order to carry out the foregoing steps, the following field parties are required: (a) transit party, (b) leveling party, (c) topography party, (d) land-line party, (e) cross-sectioning party, (f) bridge and masonry party, (g) resurvey party.

General Requirements,—Each party should work with snap and vigor and accomplish the best results practicable, both as to quality and quantity. To this end each member of the party should not only be careful, exact, and rapid in the discharge of his own duties, but avoid interfering with the work of others, such as obstructing the view of the transitman. In order to give each student practice in all the positions, the posts will be shifted daily, progressing to the higher positions in the party. The student should not underrate his practice in the subordinate positions, nor fail to make proper use of his more responsible duties. The usual decorum of field parties will be observed.

TRANSIT PARTY.—It is the duty of the transit party to establish the traverse line upon which to base the levels and topography. The student transit party will consist of the following members: (1) chief of party, (2) transitman, (3) head chainman, (4) rear chainman, (5) stakeman, (6) axeman, (7) front flagman, (8) rear flagman. The duties and equipment of the respective members are stated below.

Chief of Party.—(Party list, map of line, 50-foot metallic tape, railroad curve text book.) The chief of party is responsible for the general progress and quality of the work. It is his duty to direct the survey; see that each man does his work properly and with sufficient accuracy and despatch; check the transitman's work when necessary; keep the transit notes if the transitman is pushed; and make himself generally useful. He should be thoroughly acquainted, before going to the field, with the situation and with the data applicable to the work of the day. In requiring subordinate members of the party to perform their work properly, he should carefully preserve the dignity of his own position. Should there be no chief, these duties will be shared by the transitman and head chainman under the former's directions.

Transitman.—(Transit, reading glass, adjusting pin, transit note book, railroad curve text book, figuring pad.) The transitman runs the transit, keeps the notes, and in the absence of the chief, directs the work of the party. He should do careful and exact as well as rapid work, since the progress and character of the survey are usually controlled chiefly by the skill of the transitman.

In leveling up, keep the lower parallel plate about level. Avoid undue tightness of foot screws. In setting the vernier to zero, use a quick converging motion with the tangent movement and note the adjacent graduations. If the transit has lost motion, learn which way to get the slack on the tangent screws. As a rule, use the lower motion by preference. Habitually back sight to the rear with telescope reversed, then plunge the telescope on prolongation and read the deflection right or left. If practicable, base the calculated bearings on a true meridian; otherwise, allow for the magnetic declination at a station which seems to be free from local attraction and thus obtain a reference meridian. Check all deflection angles by needle reading, both as to amount and direction. Lack of proper adjustment is no excuse for error. Always prolong a tangent line by double sightings. Also check deflection angles from time to time, by double sightings. Check on back sight before finally approving any precise point; likewise never fail to conclude the observations at each transit station by checking on the back sight. In such check it is usually best to sight back precisely on the point and then note whether the vernier has the proper reading. Assist the flagman in plumbing the pole, and always sight as near the bottom of the pole as possible. The traisitman should admonish the chainmen, etc., to keep clear of the line.

On preliminary surveys, usually let the rear chainman line in the head chainman by eye, at least for short stretches. Do not hesitate to offset or zig-zag more or less along open ground to gain time. A rapid method for passing through heavy timber is to zig-zag on slight deflection angles right and left, tabulate the lengths in stations and deflections in minutes, and the products of the two in separate columns on the right hand page. The original line is regained by making the algebraic sum of the products zero, and the original direction is resumed by turning off a deflection which balances the deflection angle columns.

On location, each stake should be lined in carefully by transit. Small obstructions, such as trees, may be passed by parallel lines, using offsets of one foot or so at two hubs a few stations apart; the line is resumed in like manner. Where plate readings are used in rectangular or other offset methods, no sights shorter than 50 feet should be used. The equilateral triangle one station or more on a side is often used. Obstructions on curves may usually be passed readily with the aid of tables of long chords and mid-ordinates.

Curve index-readings should be calculated as though the entire curve were to be run in from the P. C.; starting with the index-reading of P. C. always equal to zero, check the calculations by noting that the index of M. C. is ¼ I, and of P. T. is ½ I. In using the notes, remember that with the transit at any point whatever on the curve the following rules apply: (1) When pointing to any station, the vernier must always be set to read the index-reading for that station; and (2) when pointing on tangent at any station, the vernier must be set to read the index-reading for that station. As a rule, the best program in curve location is: Having P. I. located, (1) measure I and assume D; (2) calculate T and E; (3) establish P. T. by chaining off T on front tangent; (4) establish M. C. by laying off E on bisecting line; (5) locate P. C. by interpolating hub at calculated station number on back tangent; (6) move transit to P. C. and fore sight on P. I.; (7) calculate curve notes (if not already done); (8) check sight on P. T. and M. C. and if satisfactory, (9) run in curve, checking for distance and angle on M. C. and P. T., moving transit ahead if desirable or necessary; (10) set up at P. T. and resume front tangent. One minute is the limit of allowable error in any curve. Mistakes in calculations or in measurements of angles will be counted serious errors. On final location the curves will be spiraled. After the line is located, reference out P. C., P. T., and other important hub points by two intersecting lines and take careful notes of the same (see method (g), Fig. 5, Chapter II.)

The transit notes should be reliable, complete, neat and distinct. Each entry should have but one reasonable meaning and that the correct one. Record station numbers from the bottom upwards, usually with ten stations per page. Repeat the last station at the top of the next page. Allow two lines per station so as to provide for sketching at 200 feet to the inch. On the middle line of the right hand page

(TRANSIT NOTES FOR RAILROAD PRELIMINARY SURVEY.)

(Organization of Party.)

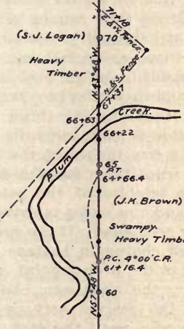
Sta.	Defl.	Mag. Ang.	Mag. B.S.	Mag. F.S.	Cal. Brg.	
86						
85						
+93	25°17'L	25°20'L	N48°30'E	N23°10'E	N26°42'E	(Var. 3°32')
84						
83						
82						
81 ●	8°57'R	9°00'R	N39°30'E	N48°30'E	N31°39'E	(Var.=3°28)
80						
79 ●	49°13'R	49°10'R	N9°20'W	N39°50'E	N43°02'E	(Var.=3°12')
						(Wire fence)
78						
77						
76						
+86.4	14°42'L	14°40'L	N4°35'E	N9°45'W	N6°11'W.	(Var. 3°34')
75						
					(N8°31'E)	→(Calc. Brg at last deflection point.)
74						

Note. Magnetic Back Sight is needle reading on back tangent prolonged.

(Calc. Bearings are based on a true meridian.)

(TRANSIT NOTES FOR RAILROAD LOCATION SURVEY.)

Sta.	Curves	Index.	Defl.	Calc. Brg.	Needle.
71	(Repeat this sta. bottom next pages)				
70					
69 ⊙					
68					
67					
66					
65					
+66.4	P.T.	7°00'	14°00'R	N43°48'W	N47°25'W.
+50		6°40'	0°20'−		
64		5°40'	1°00'		
+50	D = 4°00', I = 14°00'R, T = 175.9, L = 3.50	4°40'	1°00'		
63		3°40'	1°00'		
+50		2°40'	1°00'		
62		1°40'	1°00'		
+50		0°40'	1°00'		
+16.4	P.C.	0°00'	0°40'+	N57°48'W	N61°20'W
61					
60					
59 ⊙	(Repeated from top preceding pages)				



mark each station with a dot and number every fifth station which should also be enclosed in a circle. The transit notes should include sketches of prominent land and street lines, stream crossings and other prominent topographic details, with pluses shown in the sketch. The notes should include date, weather, organization of party, etc. An appropriate title page giving name of survey, date of commencement and completion, etc., should be prepared. The notes will be kept in the prescribed form. The field notes are to be returned at the close of the day's work. All estimated data should be noted as such.

Completeness and neatness of notes and records, facility and accuracy in handling the instrument, and promptness in advancing the progress of the survey will count in the estimate of the work of the transitman.

Head Chainman.—(Flag pole.) The progress of the chaining depends chiefly on the activity of the head chainman. After setting a stake he should move off briskly (preferably at a trot) and be prepared for the "halt" signal as he approaches the next station. When the full chain length is pulled out, the head chainman turns, holding the flag pole in one hand and the chain handle in the other, and sets the pole in line by signal from the rear chainman or transitman. Much time can be saved in this process if the head chainman habitually walks about on line and if he sights back over the two stakes last set. If on curve location, he should line himself in on the prolongation of the preceding station chord, and then offset by pacing or with flag pole a distance in feet equal to 1¾ times the degree of the curve; the calculation is made mentally and the pole can usually be set within a few inches of the correct position by the time a speedy transitman has the deflection angle set off. Having the line established, the pole is shifted to the correct distance, and the stake is driven plumb in the hole made by the flag pole spike. If the survey is a rapid preliminary line, the head chainman hastens ahead the instant the stake is started at the proper point, although in a more careful preliminary the chainmen check the distance to the driven stake. On location surveys it is customary for the chainmen to wait until the stake is driven and mark the exact distance on the top of the stake with the axe blade, and the exact line by signal from the transitman. In this process, the head chainman should keep in mind the convenience of the transitman, and in case the line is being run to a front flag, the chainman should be careful to clear the line frequently to allow check sights ahead. In breaking chain on steep slopes the full length of chain should usually be pulled out ahead and the chain thumbed at the breaking points so as to avoid blunders; a plumb bob or flag pole should be used in the process. In passing over fences it often saves time to drive a 10-d nail, with "butterfly" attached, in the top plank to serve as a check back sight from the next transit point. The chainmen should carefully avoid obstructing the transitman's view, to which end they should walk on the outside when locating curves.

Rear Chainman.—(100-foot chain or tape, chaining pins (if allowed), figuring pad or note book.) As the rear chainman approaches the stake just set, he calls out "halt" and holds the end of the chain approximately over the stake, quickly lines in the flag pole in the hand of the head chainman (or the pole is lined in by the transitman), the precise distance is given, and the chainmen move on briskly. As a rule, pluses should be read by the rear chainman, the front end being held at the point to be determined. Fractions will usually be taken to the nearest 0 1 foot, although 0 01 foot may at times be properly noted. It is the duty of the rear chainman to keep a record of pluses and topographic details when the transitman is not at hand. This record may be kept on a figuring pad and the memoranda handed at the first opportunity to the transitman, who transfers the data to his book and carefully preserves the slips for future reference. It is usually better, however, to keep the auxiliary notes in a memorandum book instead of on the loose slips. The chainmen should carefully avoid disturbing the transit legs.

The responsibility for correct numbering of the station stakes rests chiefly on the rear chainman. It is his duty to remember the number of the previous station so as to catch blunders on the part of the stakeman. As he reaches the stake just driven, he mentally verifies its number and repeats it distinctly for the guidance of the stakeman in marking the stake to be driven; the stakeman responds by calling the new number, and each repeats his number as a check before final approval. The rear chainman then charges his mind with the numbers and checks the newly set stake on reaching it. In case of doubt he returns to the preceding stake and notes its number.

Stakeman.—(Sack of flat and hub stakes, marking crayon, handaxe.) The stakeman with his supply of flat and hub stakes in a sack, should keep up with the head chainman and be standing, with stake and marking keel in hand, ready to number the new station stake on hearing the rear chainman call out the preceding station number; the numbering is repeated, as already explained, before the stake is driven. Chaining pins are not used, but their equivalent in checking tallies may be had by numbering the stakes ahead and tieing them up in sets of ten. By numbering stakes at slack moments the stakeman gains time to assist the axeman in clearing the line, etc. However, special care should be taken to avoid omissions and duplicates. The stakeman should finish numbering the stake and hand it to the axeman by the time the head chainman has fixed the exact station point. The stakes should be numbered in a bold and legible manner, the keel being pressed into the wood for permanency. The number should read from the top of the stake downward. Stakes on an offsetted line should be so marked, as 4′L or 2′R, beneath the station number. When survey lines are lettered, the serial letter should precede the station number. Guard stakes for P. I., P. C., P. T., reference points (R. P.), etc., should be clearly marked. The stakeman should assist the axeman in clearing the line and should drive stakes when the axeman is delayed. He should carefully avoid obstructing the transitman's view. The stakeman is under the direction of the head chainman.

Axeman.—(Axe, tacks, (and if so instructed) an extra sack of stakes with marking keel.) It is the duty of the axeman to drive stakes, remove underbrush from the line, clear an ample space about the transit station, etc. He is expressly warned, however, in student field practice, not to hack or cut trees or damage other property in any way, and in general, not to trespass on the rights of owners of premises entered in the progress of the survey.

The flat station stakes are driven firmly crosswise to the line with the numbered face to the rear. Hubs are driven about flush and usually receive a tack; they are properly witnessed by a flat guard stake driven 10 inches or so to the left, the marked face slanting towards the hub, as shown in Fig. 9, Chapter II. The axeman receives the marked stake from the stakeman and drives it plumb at the point marked by the spike of the flag pole. On location or careful preliminary surveys when the stakes are being lined in by transit, the axeman should stand on one side when driving and keep a lookout for signals from the transitman. In shifting the stake as signaled he should use combined driving and drawing blows with the axe. When the precise point comes much to one side of the top of the hub, another hub should be driven alongside and the first one driven out of sight before the tack is set. The axeman should move ahead briskly and avoid delay to the chaining. The stakeman should, when necessary, drive the stake with the spare handaxe. When the field force is scant, one man may serve in both capacities. The axeman is under the direct charge of the head chainman.

Front Flagman.—(Flag pole, small supply of hubs and guard stakes in stake sack, handaxe, a few 10-d nails.) It is the duty of the front flagman to establish hub points ahead of the chaining party under the direction of the chief and transitman. In selecting transit stations he should keep in mind visibility and length of both fore sight and back sight, and to this end, points should be taken on ridge lines and where underbrush, etc., is least in the way. The practice of planting the flag pole behind the hub may be warranted occasionally, as for example, when the field party is shorthanded, but never when the regular flagman is not specially detailed for other duties. The front flagman should keep close watch on the transitman and should habitually stand with the spike of the flag pole on the tack head and plumb the pole by standing squarely behind it and supporting it between the tips of the fingers of the two hands. Should the front flagman be flagging for an interpolated point depending on a foresight which his pole would conceal, he should clear the line for a check sight by leaning the pole to one side. When crossing fences he should, when convenient, establish check sights on the top plank by driving a spike and attaching a "butterfly."

Rear Flagman.—(Flag pole, hatchet, slips of paper.) The rear flagman gives back sight on the preceding transit station. The details of his duties are much the same as those of the front flagman. It is an excellent plan for him to cut a straight sappling or limb and plant it exactly behind the hub when signaled ahead. This picket pole is made more visible by splitting the top and inserting a slip of paper, to make a "butterfly." A series of such pickets on a long tangent line often affords a fine check on the work when an elevated transit point is reached.

LEVEL PARTY.—It is the purpose of the level party to secure data concerning the elevations of the points along the line so that an accurate profile may be made and the grade line established. The leveling party should be on the alert to detect errors in the work of the transit party, such as omitted or duplicated stations, etc. The party consists of two members: (1) leveler, (2) rodman. In very brushy country an axeman may be added, but this is usually unnecessary if the line cleared by the transit party is followed.

Leveler.—(Level, adjusting pin, level note book.) The leveler should follow the most approved methods described under the head of differential and profile leveling in Chapter IV. The nearest 0.01 foot should be observed on turning points and bench mark rod readings and elevations and on occasional important profile points. The fore sight rod readings on ground profile points are to be taken only to the nearest 0.1 foot and the nearest 0.1 foot in the height of instrument is to be used in calculating the elevation. (Beginners sometimes calculate elevations to 0.01 foot when the rod readings are taken only to the nearest 0.1 foot.) The leveler should be rapid with his level as well as with figures. He should calculate elevations as fast as the rod readings are taken and should systematically check up the turning point and instrument heights as the work proceeds. As results are verified the same should be indicated by check marks. Each page of notes should be checked by summing up turning point back and fore sight rod readings, and comparing their difference with the difference between the first and last elevations or instrument heights, as the case may be, on the page. Follow the prescribed form. As far as possible, bench marks should be checked by including them in the circuit as turning points. Balance back and fore sight distances on turning points. Permanent bench marks should be established at least every 1500 feet, and located in places at once convenient and free from disturbance

(LEVEL NOTES FOR RAILROAD SURVEY.)

S	+	⊼	−	E	⊙	Oct. 13, 1893. Cool.	Keen, Leveler Swift, Rodman.
B.M.					712.39	Spike in notch at root of Elm tree, 68′R of	
⊼	+7.21	719.60				Sta. 15+48, 2′S. of rail fence.	
16			8.4	711.2		Ground.	
17			7.2	712.4		"	
18			5.4	714.2		"	
19			6.4	713.2		"	
20			4.5	715.1		"	
+50			2.1	717.5		"	
21			0.2	719.4		"	
⊙			−0.15		719.45	On hub at Sta. 21.	
⊼	+8.83	728.28					
22			8.4	719.9		Ground.	
+28			6.6	721.7		" P.C. 1°00′ C.-R.	
23			4.8	723.5		"	
24			3.8	724.5		"	
25			3.7	724.6		"	
26			1.6	726.7		"	
⊙ B.M.			−1.57		726.71	Top of granite boulder. 74′R. Sta. 26+17.	
⊼	+8.92	735.63					
27			5.6	730.0		Ground.	
+32			5.7	728.9		" P.T.	
28			3.8	731.8		"	
29			3.7	731.9		"	
30			4.3	731.3		"	
31			5.2	730.4		" (Checked ⊙'s, B.M.'s and H.I.'s with	
	+24.96 − 1.72 +23.24	735.63 712.39 +23.24	−1.72 (Check)	Profile data above.		Rodman's Peg Book.)	

during construction. Later levels should check within 0.05 foot into the square root of the length of circuit in miles. When a discrepancy is found, a line of check levels must be run to fix responsibility for the error. In crossing streams, secure high water elevations, with dates, especially of extraordinary floods, also low water level. In crossing highways obtain elevations each side for some distance with a view to avoid grade crossings. In going up or down steep slopes, gain all the vertical distance possible each setting, and follow a zig-zag course. The bottom of deep gullies may be determined by hand level. Assist the rodman in plumbing the rod, and on turning points and benches have the rod gently swung in a vertical plane to and from the instrument and take the minimum reading. The self-reading rod is to be preferred. Many levelers use the Philadelphia rod without target. If the target is used on turning points, the leveler should check the rod reading when practicable.

Completeness, correctness and neatness of notes and records, and facility and accuracy in handling the level will be given chief weight in fixing the merit of the leveler's work. The level notes are to be returned at the end of the day's work.

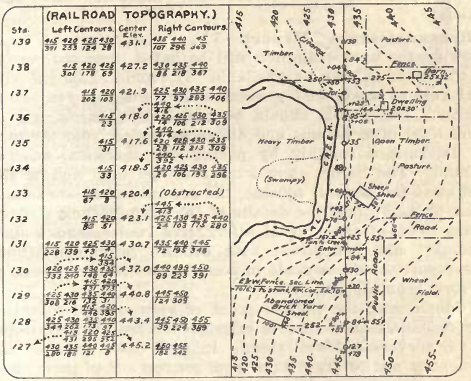
Rodman.—(Leveling rod, peg book, hatchet, turning point pegs, spikes, keel.) The rodman holds the rod at station stakes and at such plus points as may be required to make a representative profile. It is his duty to identify each station point and be on the lookout for duplicated or omitted stations. To this end he should habitually pace in each station, especially in grass or underbrush, and call out or signal the station number to the leveler. Should a blunder in station numbering appear, he should positively confirm the fact by retracing several stations, and then carry the corrected stationing ahead. The rod should be held truly plumb, which is best done by standing squarely behind the rod and supporting it with the tips of the fingers of both hands. On turning points, the rod should be waved gently in a vertical plane to and from the instrument. The rodman should pay special attention to placing the target right for long rods and examine it to note if it has slipped before reading the rod. Errors of 1 foot, 0.1 foot, etc., should be carefully guarded against. Turning points should be selected with special reference to their solidity, and care should be taken not to disturb them. Station pegs and hubs are often used for turning points; when so used, the precise fore sight to 0.01 foot should follow the usual ground rod reading to the nearest 0.1 foot. The rodman should use good judgment in selecting bench marks, locating them out of reach of probable disturbance during construction and describing them so as to be easily found. He should be active and do his best to keep close up with the transit party. The rodman should keep a peg book for recording turning points and instrument heights, and check his computations independently and compare results with the leveler.

TOPOGRAPHY PARTY.—It is the purpose of the topography party to secure full data for mapping contours, property lines, buildings, roads, streams, and other important topographic details. The width of territory to be embraced in the survey depends on local conditions; in places it may be as much as one-fourth or one-half mile from the line, although it is usually better to run alternate lines when the distance to be included becomes so great. The topography party often consists of only two men, but a party of four is much more efficient. Sometimes no regular topography party is provided, but after running a few miles of line ahead, the transit and level parties are formed into several parties to bring the topography up to the end of the preliminary line. For student practice the topography party will consist of four members: (1) topographer, (2) assistant topographer, (3) topography rodman, (4) tapeman.

Topographer.—(Topography board, topography sheet (or several sheets), hard pencil, compasses, eraser, etc.) The topography sheet should be prepared before going to the field, showing the alinement and other data needed from the transit notes, and elevations of all stations and pluses from the level notes. Cross-section paper is to be preferred. The center line may be plotted to one side of the center line of the sheet, when the topography is to be taken farther in one direction than the other. In order to secure full details, the scale of the field plat may well be double (or even more) that of the finished map. The topography sheet should show local conditions, such as gravel banks, rock ledges, etc., suitable for ballast or other constructive use; out-croppings of rock or other material which may affect the classification of the graduation; character of substrata at sites of bridge or other masonry work; springs, wells, streams, etc., suitable for water supply; approximate flood levels and other data relating to waterways or surface drainage; location of streams, especially with reference to desirable crossings, freedom from probable change of channel, etc.; location of highways including elevations some distance either way with special reference to avoiding grade crossings; other railroad lines, with the same point in view; character and condition of crops and other farm improvements, names of owners, etc.,—in short, any and all information that is at all likely to be of service in mapping the route, projecting the location, during construction, etc. In locating a group of buildings some distance from the line, fix the principal one by tie lines, by intersection or polar coordinates, and the others by measurement and sketch from it. Locate buildings near the line by rectangular offsets, or by intersections of the principal outlines with the survey line. Contours are located by means of the hand level used by the assistant topographer. The contour interval should be five feet ordinarily, but may be increased to ten or more feet on very steep slopes. The contour data should be selected with special reference to ridge and gully lines (see problem and plat on contour leveling, Chapter IV.) Ordinarily hand level lines may be run out at right angles; angling lines along gulches and ridges may be located by estimation, pocket compass or tie lines. The plat is made by the topographer from data collected by the other members of the party. A common fault with the beginner in such work is the omission from the plat of important numerical data, such as station numbers of land-line crossings, etc., owing to an undue attention to the minute details of the drafting work. A good topography record with contour notes on the left hand page and field sketch showing all numerical data on the right, is shown in the accompany form.

Assistant Topographer.—(Hand level, pocket compass, topography note book.) It is the duty of the assistant topographer to collect data for the use of the topographer in making the plat. He uses the hand level, notes station numbers, distances, bearings, etc., and makes such record of the same as may be required to fit local conditions. In contouring, a special rod with adjustable base (see Fig. 19, Chapter IV.), if available, may be used; otherwise, an ordinary flag pole with alternate feet red and white is employed. Beginning with the known profile elevation, as extracted from the leveler's record, even five-foot contours are located, as a rule, nominally every 200 to 500 feet at right angles to the line, except as ruling ridges or gullies may suggest other directions. His record should be ample and legible, and include data and information which may not properly be placed on the plat. All estimated elevations, distances or dimensions should be noted as such. The assistant topographer works under the direction of the topographer, but is expected to take the initiative in the collection of data so as to permit his superior to devote proper attention to the field plat.

Topography Rodman.—(Topography rod with adjustable base (see (f), Fig. 19, Chapter IV.) or flag pole, hatchet.) It is the duty of the rodman to hold the topography rod as directed by the assistant topographer. He should be active and continually on the alert for information or data which the record book or sheet should contain. The rodman holds



the zero end of the tape in measuring the distances. He should acquire skill in pacing on rough as well as smooth ground, and when sufficiently exact especially on ground remote from the surveyed line, he should gain time by pacing in the distances to contour lines.

Tapeman.—(Metallic (or band) tape, set of chaining pins, flag pole.) It is the duty of the tapeman to determine distances with the help of the rodman. He should be vigilant in checking up tallies, reading fractions, leveling the tape, breaking chain, plumbing down ends, etc., and should never be the cause of needless delay in the work. When required, he should measure angles, take tie lines, etc., with the tape.

OFFICE WORK.—The office work of each student includes; (1) reconnaissance map, profile and report; (2) map showing preliminary lines with topography and projected location lines; (3) preliminary profile with grade lines, approximate estimate of quantities, etc.; (4) final location map (traced from preliminary map); (5) location profile; (6) copies of field notes; (7) cross-section notes and estimate of graduation quantities; (8) estimate of cost of construction; (9) monthly estimates, progress profile, haul, prismoidal and curvature corrections, vouchers, etc., final estimate.

Reconnaissance Report.—The reconnaissance map showing the area examined will be based upon such maps of the route as may be available. It should show the several ruling points and general routes selected for actual survey. The profile should be based upon barometric or hand level observations and distances scaled from the map or determined roughly by pacing or otherwise on the ground. The report should refer to the map and profile and state the general scheme, the several ruling considerations or conditions, the details of the examination, a rough comparison of the several alternative routes, and a final summary and conclusion with definite recommendations. The report should be made in accordance with best usage as to form, composition, etc.

(Considering the limited point of view of the beginner, the reconnaissance reports may not be required until the actual surveys are well along. In such case, however, the student is not to draw data from sources other than those above outlined.)

Preliminary Map.—The mapping should be the best product of the student's skill as a draftsman, and should conform closely to the department standards, which are based upon best current usage of leading American railroads. Unless otherwise instructed, the preliminary map will be made on eggshell or paragon paper. There are three ways to plot the skeleton of the preliminary survey: (1) by laying off each successive deflection angle and distance from the preceding line; (2) by laying off the successive calculated courses and distances from a precisely drawn meridian or other reference line; and (3) by rectangular coordinates. The first method should not be used, since cumulative errors are probable. The second is rapid and free from serious objection; if preferred, a modified base line may be assumed and the calculated bearings transferred to the same; the angles may be laid off by means of scale and table of natural trigonometric functions from a precisely drawn base line and then transferred, as required, by parallel ruler or triangle; this method is used most in practice. The third method is the most exact, and will be used by the student unless the second is specified. It involves the calculation of

PLOTTING SHEET, PRELIMINARY LINE "A".

	Sta.	Defl.	Cal. Brg.	Dist. Ft.	Latitudes N.	Latitudes S.	Departures E.	Departures W.	Tot. Lat. N.	Tot. Dep. W.	
a	0		N32°34'W	2489.1	2097.7			1339.8	0.0	0.0	a
b	$24+89^{1}$	32°07'L	N64°41'W	78.5	33.6			71.0	2097.7	1339.8	b
c	$25+67^{5}$	28°15'R	N36°26'W	464.1	373.4			275.6	2131.3	1410.8	c
d	$30+31^{7}$	10°38'R	N25°48'W	225.5	203.0			98.1	2504.7	1686.4	d
e	$32+57^{2}$	2°27'R	N23°21'W	436.7	400.9			173.1	2707.7	1784.5	e
f	$36+93^{2}$	18°40'R	N4°41'W	164.8	164.2			13.5	3108.6	1957.6	f
g	$38+58^{2}$	46°59'L	N51°40'W	152.9	94.8			119.9	3272.8	1971.1	g
h	$40+11^{6}$	37°05'R	N14°35'W	176.0	170.3			44.3	3367.6	2091.0	h
i	$41+87^{6}$	16°07'R	N1°32'E	310.1	309.9		8.3		3537.9	2135.3	i
j	$44+97^{7}$	21°27'L	N19°55'W	105.9	99.6			36.1	3847.8	2127.0	j
k	$46+03^{6}$	6°17'L	N26°12'W	307.8	276.2			135.9	3947.4	2163.1	k
l	$49+11^{5}$	75°06'L	S78°42'W	331.9		65.0		325.5	4223.6	2299.0	l
m	$52+43^{5}$	73°14'R	N28°04'W	202.7	178.9			95.4	4158.6	2624.5	m
n	$54+46^{4}$	21°29'L	N49°33'W	156.4	101.5			119.0	4337.5	2719.9	n
o	$56+02^{6}$	32°05'L	N81°38'W	332.6	48.4			329.1	4439.0	2838.9	o
p	$58+35^{9}$	36°52'R	N44°46'W	308.7	219.2			217.4	4487.4	3168.0	p
q	$62+43^{7}$	11°54'L	N56°40'W	128.1	70.4			107.0	4706.6	3385.4	q
r	$63+71^{8}$	42°20'L	S81°00'W	251.8		39.4		248.7	4777.0	3492.4	r
s	$66+23^{6}$	50°05'R	N48°57'W	334.2	219.5			252.0	4737.6	3741.1	s
t	$69+57^{8}$	35°11'L	N84°08'W	266.9	27.3			265.5	4957.1	3993.1	t
u	$72+24^{2}$	2°05'R	N82°03'W	317.0	43.8			313.8	4984.4	4258.6	u
v	$75+41^{2}$	7°22'L	N89°25'W	557.8	5.7			557.8	5028.2	4572.4	v
w	$80+99^{5}$	43°15'R	N46°10'W	1067.6	739.4			770.1	5033.9	5130.2	w
x	$91+67^{1}$			9167.1	5877.7	104.4	8.3	5908.6	5773.3	5900.3	x
					104.4			8.3			
					5773.3	Check		5900.3	Check		

PLOTTING SHEET, LOCATION SURVEY, BELT R.R. EXTENSION.

From Sta.	To Sta.	Length Tangts Ft.	Length Curves Ft.	Angle I.	Degree D.	Radius R. Ft.	Tan Dist T. Ft	Dist P.I. to P.I. Ft.	Calc Bearing.	Lat / Dep. Ft.	Tot. Lat / Tot. Dep. Ft.
Tangent	Main	Track,	A.B.&C. R.R.						S35°15'E	P.I. =	0.0 0.0
P.C. 0	P.C.C. 1		100.0	7°30'R	7°30'	764.1	50.1	1320.4	(S27°45'E) S/E		
P.C.C. 1	P.T. 26		2500.0	25°00'R	1°00'	5730.0	1270.3			P.I. =	S/E
P.T. 26	P.C. $43+55^{6}$	1755.6						3963.7	S 2°45'E S/E		
P.C. $43+55^{6}$	P.T. $61+68^{1}$		1812.5	36°15'R	2°00'	2865.0	937.8			P.I. =	
P.T. $61+68^{1}$	P.C. 93+65	3196.9						4580.1	S33°30'W S/W		
P.C. 93+65	P.T. $102+51^{7}$		886.7	15°18'L	1°30'	3820.0	445.4			P.I. =	
P.T. $102+51^{7}$	P.C. 143+90	4138.3						5185.7	S20°12'W S/W		
P.C. 143+90	P.T. $155+76^{7}$		1186.7	23°44'L	2°00'	2865.0	602.0			P.I. =	
P.T. $155+76^{7}$	P.C. 170+43	1466.3						2618.3	S3°52'E S/E		
P.C. 170+43	P.T. $181+29^{7}$		1086.7	21°44'R	2°00'	2865.0	550.0			P.I. =	
P.T. $181+29^{7}$	P.C. $184+50^{6}$	320.9						1317.3	S18°12'W S/W		
P.C. $184+50^{6}$	P.T. $193+19^{5}$		868.9	13°02'R	1°30'	3820.0	436.4			P.I. =	
P.T. $193+19^{5}$	P.C. $213+06^{4}$	1986.9						2816.9	S31°14'W S/W		
P.C. $213+06^{4}$	P.T. $220+90^{8}$		784.4	11°46'L	1°30'	3820.0	393.6			P.I. =	
P.T. $220+90^{8}$	P.C. $230+34^{6}$	963.8						1963.8	S19°28'W S/W		
P.C. $230+54^{6}$	P.T. $242+49^{6}$		1195.0	23°54'R	2°00	2865.0	606.4			P.I. =	
P.T. $242+49^{6}$	P.C. $252+35^{9}$	986.0						1861.3	S43°22'W S/W		
P.C. $252+35^{9}$	P.T. $257+52^{7}$		517.1	36°47'L	7°30'	764.1	268.9			P.I. =	
		14814.7	10938.0	127°25'R	Tangt Main Trk	5560.9		25627.5	S4°35'W		(Check
		10938.0		87°35'L	X.Y.&Z.R.R.	× 2			See		last
	Check	25752.7		39°50'R	(1stT + lastT) =	11121.8			Check		totals
				S35°15'E		−319.0			in 5th		by Σ)
					Σ Tangts. =	10802.8			Column.		
						14814.7					
			Check.	S4°35'W	Check.	25617.5					

(Last two columns are to be filled in and checked by each student.)

a plotting sheet, as shown in the accompanying form. The axis is usually a meridian line, but any line may be taken and the courses changed to suit. In making the plotting table, the data, calculated bearings, distances, etc., should be carefully checked through to the last point in the skeleton before the plotting is begun. Only one axis should be plotted, preferably the one having greater totals, so as to give short perpendiculars. Starting from the origin, 1000-foot points are pricked in along the axis to the specified scale, and marked 0, 10, 20, etc.; the totals are interpolated on the axis and lettered; exact perpendiculars about the right length are erected; the second point is established by scaling the perpendicular and the line is checked back on the preceding point; if correct, the stations are pricked in and every fifth station and deflection points are enclosed in a small circle and neatly numbered; the next course is so located and checked back by length of hypothenuse, the stations fixed and numbered, and so on to the end of the line; the courses should be taken in their order and none passed without checking satisfactorily. After the skeleton is completed, the topographic details are penciled in, and the map finished and inked. The title, border, meridian (both true and magnetic), etc., should be first-class in quality and in keeping with the rest of the map. Crude or careless lettering or other details of the map will cause its rejection. The title of the map, profile, etc., should be given in brief on the outside of the sheet or roll at each end.

Preliminary Profile.—Use Plate A profile paper in making the profiles. The level notes should first be carefully verified and then one person should read off while another plots the data. A hard pencil, 6H or 7H, sharpened to a long needle point should be used. The stations are first numbered along the bottom from left to right (or the reverse, as prescribed); leaving six inches or so at the left for a title, and beginning at a prominent line with station 0, every tenth station is so numbered. The notes are examined for lowest and highest elevation and a prominent line is assumed as an even 50 or 100-foot value relative to the datum. The horizontal scale is 400 feet and the vertical scale 20 feet to the inch. Points should be plotted no heavier than necessary, since the surface of profile paper will not permit much erasing. The surface line should be traced in close up to the plotted points, owing to the danger of overlooking abrupt breaks such as streams, ditches, etc. Pluses should be fixed by estimation. The surface line when completed should be inked with a ruling pen used freehand; the weight of the line should be about the average of the ruled lines on the profile paper. (A special profiling or contouring pen is much used for this purpose.) The profile should show the grade line, grade intersection, elevations and rates of grade in red; water levels, and data relative to same in blue; surface line, station numerals, etc., in black; the alinement, important land lines, streams, etc., should be shown at the bottom of the profile in black. The grade line should be laid nominally with a view to balance the cut and fill quantities, but this should be varied to suit local conditions, such as drainage, the elimination of grade crossings, classification of materials, etc. The maximum gradients, the rate of compensation for curvature, etc., will be made to suit the specified conditions. The compensation for curvature will be allowed for on the preliminary profile by dropping the grade line on maximum gradients at each deflection point. Grade intersection elevations and rates of grade will be given to the nearest 0.01 foot.

Approximate Estimates.—Rapid estimates of earthwork quantities may be made direct from the profile either by reference to a table of level sections, or preferably by means of an earthwork scale, shown in the accompanying diagram. This scale is graduated in hundreds of cubic yards for the particular roadbed base and side slopes. The data for making the scales are given in the table. The quantities may be jotted down for addition or lumped mentally, or an adding strip may be inserted in slits near one edge of the scale. In using this scale it is customary to make no deduction for minor waterways. Estimates made in this way from the profile of a careful preliminary survey, often do not vary more than five per cent from the final construction quantities.

Location Map.—The location map may be traced from the preliminary map and should include the topography and such details as usually appear on the final record map of the located line. Contour lines may be traced in cadmium yellow to insure satisfactory blue printing.

Location Profile.—The location profile should be exe-

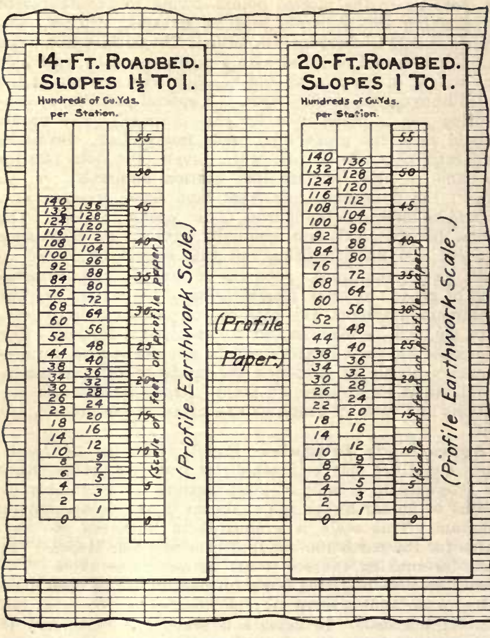


Fig. 39.

cuted according to the standard specimen, and should include estimates of earthwork as determined from the actual cross-section notes, and quantities of other construction materials. Curvature compensation will be shown on the location profile by reduced maximum gradients. Vertical curves will be calculated at a rate of change not to exceed 0.05 foot per station, except at summits where it may be 0.10 foot or more per station. It should be prepared as

CENTER CUT OR FILL IN FEET FOR GIVEN QUANTITIES PER STATION. (DATA FOR EARTHWORK SCALE.)

Cubic Yards Per 100 Feet	Side Slope, 1 to 1. Width of Roadbed in Feet.					Side Slope, 1½ to 1. Width of Roadbed in Feet.					Cubic Yards Per 100 Feet
	14	16	18	20	22	14	16	18	20	22	
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0
100	1.7	1.5	1.4	1.3	1.2	1.6	1.5	1.4	1.2	1.0	100
200	3.1	2.8	2.6	2.4	2.2	2.9	2.6	2.5	2.3	2.0	200
300	4.4	4.0	3.7	3.5	3.2	4.0	3.6	3.5	3.2	2.9	300
400	5.5	5.1	4.7	4.4	4.4	5.0	4.7	4.4	4.1	3.6	400
500	6.6	6.1	5.7	5.3	5.0	5.9	5.6	5.2	4.9	4.4	500
600	7.5	7.0	6.6	6.2	5.8	6.7	6.3	6.2	5.6	5.1	600
700	8.4	7.9	7.4	7.0	6.6	7.5	7.1	6.7	6.2	5.8	700
800	9.3	8.7	8.2	7.8	7.4	8.2	7.8	7.4	6.9	6.4	800
900	10.1	9.5	9.0	8.5	8.0	8.9	8.5	8.1	7.5	7.0	900
1000	10.9	10.3	9.7	9.2	8.7	9.5	9.1	8.7	8.2	7.8	1000
1200	12.3	11.7	11.1	10.6	10.1	10.8	10.3	9.9	9.4	8.8	1200
1400	13.7	13.0	12.4	11.9	11.3	11.9	11.4	11.0	10.4	9.7	1400
1600	14.9	14.2	13.6	13.1	12.5	12.9	12.5	12.0	11.4	10.7	1600
1800	16.2	15.5	14.8	14.2	13.6	13.9	13.4	13.0	12.4	11.7	1800
2000	17.3	16.6	15.9	15.3	14.7	14.9	14.4	13.9	13.2	12.6	2000
2200	18.2	17.6	17.0	16.3	15.7	15.8	15.2	14.8	14.1	13.5	2200
2400	19.4	18.7	18.0	17.3	16.7	16.6	16.1	15.6	14.9	14.3	2400
2600	20.4	19.7	19.0	18.3	17.7	17.5	17.0	16.5	15.8	15.1	2600
2800	21.4	20.5	19.9	19.3	18.6	18.3	17.8	17.2	16.5	15.8	2800
3000	22.3	21.5	20.8	20.2	19.5	19.0	18.5	18.0	17.3	16.6	3000
3200	23.2	22.4	21.7	21.0	20.4	19.8	19.2	18.8	18.0	17.3	3200
3400	24.2	23.4	22.6	21.9	21.2	20.5	20.0	19.5	18.7	18.0	3400
3600	24.9	24.1	23.4	22.7	22.1	21.2	20.7	20.2	19.4	18.6	3600
3800	25.8	25.0	24.3	23.6	22.9	21.9	21.4	20.8	20.0	19.3	3800
4000	26.4	25.8	25.1	24.4	23.6	22.6	22.0	21.5	20.7	20.0	4000
4400	28.2	27.4	26.6	25.9	25.2	23.9	23.3	22.8	22.0	21.2	4400
4800	29.7	28.9	28.1	27.4	26.8	25.1	24.6	24.0	23.2	22.4	4800
5200	31.1	30.3	29.5	28.8	28.1	26.3	25.7	25.2	24.4	23.6	5200
5600	32.5	31.7	30.9	30.1	29.4	27.4	26.9	26.3	25.5	24.7	5600
6000	33.8	33.0	32.2	31.5	30.7	28.5	28.0	27.4	26.6	25.8	6000
6400	35.2	34.3	33.5	32.8	32.0	29.6	29.0	28.5	27.6	26.8	6400
6800	36.4	35.6	34.8	34.0	33.2	30.6	30.0	29.5	28.7	27.8	6800
7200	37.7	36.8	36.0	35.2	34.4	31.6	31.0	30.5	29.6	28.8	7200
7600	38.9	38.0	37.2	36.4	35.5	32.6	32.0	31.5	30.6	29.8	7600
8000	40.0	39.2	38.3	37.5	36.8	33.6	33.0	32.4	31.6	30.8	8000
8400	41.1	40.3	39.6	38.7	37.9	34.5	33.9	33.3	32.5	31.7	8400
8800	42.2	41.4	40.6	39.8	39.0	35.4	34.8	34.2	33.4	32.6	8800
9200	43.3	42.5	41.6	40.8	40.0	36.3	35.7	35.1	34.3	33.5	9200
9600	44.4	43.5	42.7	41.9	41.1	37.2	36.6	36.0	35.1	34.3	9600
10 000	45.5	44.6	43.7	42.9	42.1	38.0	37.4	36.8	36.0	35.2	10 000
10400	46.5	45.6	44.8	43.9	43.1	38.9	38.4	37.7	36.8	36.0	10400
10 800	47.5	46.6	45.7	44.9	44.1	39.7	39.0	38.5	37.6	36.8	10 800
11 200	48.5	47.6	46.7	45.9	45.1	40.5	40.0	39.3	38.5	37.6	11 200
11 600	49.4	48.5	47.7	46.9	46.0	41.3	40.7	40.1	39.3	38.3	11 600
12 000	50.4	49.5	48.6	47.8	46.9	42.0	41.4	40.9	40.0	39.1	12 000
12 400	51.3	50.4	49.6	46.7	47.9	42.8	42.2	41.6	40.7	39.9	12 400
12 800	52.2	51.3	50.5	49.6	48.9	43.6	43.0	42.4	41.5	40.7	12 800
13200	53.1	52.2	51.4	50.5	49.7	44.3	43.7	43.1	42.3	41.4	13200
13 600	54.0	53.2	52.3	51.4	50.6	45.0	44.4	43.8	42.9	42.1	13 600
14 000	54.9	54.0	53.1	52.3	51.5	45.7	45.1	44.6	43.7	42.8	14 000

the final record profile. Approximate profiles of projected lines, determined from the contour map, with rough estimates of quantities will also be prepared, as specified.

Office Copies of Notes.—The complete level and transit notes, and topography notes as assigned, must be copied in the individual books by each student. These copies will be in pencil (or ink if so specified) and will be executed in a faithful and draftsmanlike manner according to the department standards of lettering, etc.

Estimates of Quantities.—The cross-section notes will be copied and the quantities of excavation and embankment calculated, as assigned. The cross-sectional areas will be calculated arithmetically and checked, especially on rough ground, by means of planimeter. The quantities will be calculated by average end areas, by tables, and by diagrams, so as to afford ample practice for the student in all the current methods. The estimate will also include all the other materials of construction.

Estimate of Cost.—Each student will make a detailed summary of the quantities, fix prices, and estimate the probable total cost of the work, or of the assigned section. The prescribed form will be followed. The prices should be based on local conditions as far as possible.

Construction Estimates.—Monthly estimates, estimates of haul, borrow pit estimates, classification, prismoidal and curvature corrections, progress profile, vouchers, force account, etc., and final estimate will be prepared by each student in accordance with prescribed forms and standards.

Right of Way Records.—Each student will be assigned a share of work in the preparation of right of way deeds and record maps. The following forms (from the "Engineering Rules and Instructions," Northern Pacific R. R.) will be used as models in preparing right of way descriptions.

(Through government subdivisions): "A strip, piece or parcel of land one hundred feet in width, situated in the northwest quarter of the northwest quarter of section ten, in township two north, range one west (S. 10, T. 2 N., R. 1 W.), Madison county, Montana, and having for its boundaries two lines that are parallel with and equidistant from the center line of the railroad of the ——— Railway Company, as the same is now located (and constructed.) For a

(ESTIMATE OF COST OF RAILROAD CONSTRUCTION.) (TITLE OF LINE FOR WHICH ESTIMATE IS MADE.) ITEM.	Measure	Price.	Quantity	Amount.
GRADUATION.				
1. Earth Excavation.	Cu. Yds.			
2. Earth Embankment Borrowed.	Cu. Yds.			
3. Earth Embankment Overhauled.	Cu.Yd.-Stas.			
4. Loose Rock Excavation.	Cu. Yds.			
5. Solid Rock Excavation.	Cu. Yds.			
6. Clearing.	Acres.			
7. Grubbing.	Sta.			
BRIDGING, CULVERTS, ETC.				
8. Timber in Bridges.	M.Ft. B.M.			
9. Iron in Bridges.	Lbs.			
10. Piling Driven.	Lin. Ft.			
11. Timber in Culverts.	M.Ft. B.M.			
12. Iron in Culverts.	Lbs.			
13. Vitrified Pipe.	Lin. Ft.			
14. Cattle Guards.	Each.			
15. Lumber in Road Crossings.	M.Ft. B.M.			
16. Spikes in Crossings.	Lbs.			
TRACK.				
17. Ties.	Each.			
18. Rail. (Wt. per yd.)	Long Tons.			
19. Angle Bars. (Wt. per pair.)	Lbs.			
20. Track Bolts. (Size.)	Kegs (Wt. pr. Kg.)			
21. Spikes. (Size.)	Kegs (Wt. pr. Kg.)			
22. Switch Stands and Fixtures.	Sets.			
23. Frogs.	Each.			
24. Switch Timbers.	Sets.			
25. Track Laying and Surfacing.	Miles.			
MISCELLANEOUS.				
26. Fencing.	Rods.			
27. Telegraph Line.	Miles.			
28. Buildings.	Each.			
30. Right of Way.	Acres.			
31. Engineering and Incidentals.	Per Cent.			
Total				

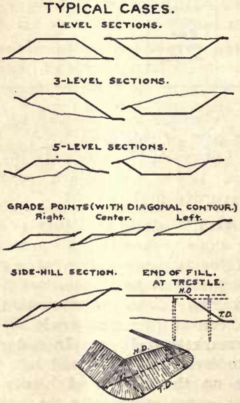
more particular description, reference may be had to the plat drawn upon and made a part of this deed."

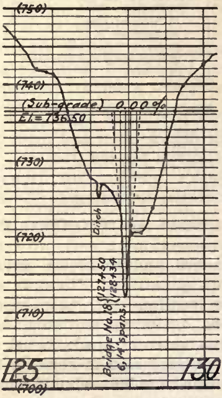
(Lots in platted tracts): "Lot seven (7), block six (6), in Smith's addition to Helena, Lewis and Clark county, Montana, according to the recorded plat thereof."

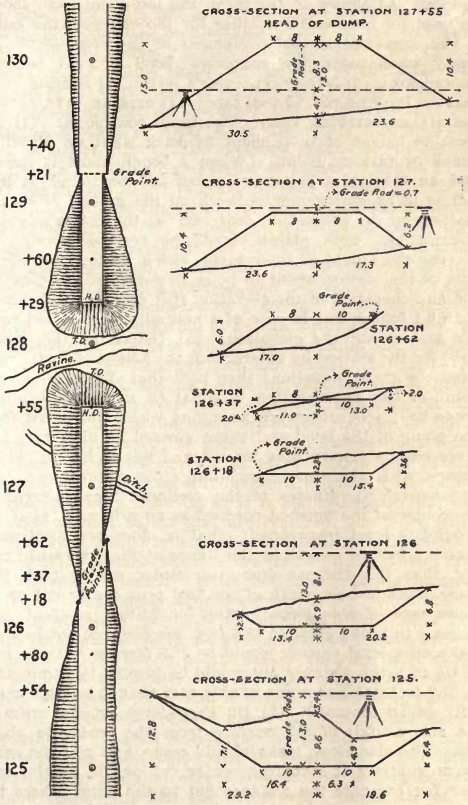
CROSS-SECTIONING PARTY.—It is the duty of the cross-sectioning party to set slope stakes for the proposed roadbed and to secure data for the calculation of earthwork quantities. The data should first be transcribed from the location level notes and profile into the cross-section book, including station numbers, surface and grade elevations, rates of grade, bench mark record, etc. In order to avoid confusion in relation to directions right and left, the station numbers should run up the page, and plenty of space left for pluses in the notes, especially on rough

(FORM FOR CROSS-SECTION NOTES.)

Sta.	Elev.	Grade.	Rate.	Surf. Rod	Grade Rod	⊼	L	C	R	Remarks.
130	742.5	736.50	0.00 % gradient Sta. 118 to Sta. 135+30	5.2	⊼747.67 11.2	⊼728.22	+5.4/16.1	+6.0	+8.0/20.0	(3-level section in cut.)
+40	739.8	736.50		7.9	11.2	F.S. 0.57 ⊙727.65	+3.3/13.0	+3.3	+3.3/13.0	(Level section in cut.)
					⊼739.61	B.S. 11.96				
+21	736.5	736.50		(3.1)	(3.1) 3.1	⊼739.61 F.S. 2.03	0.0/10.0	0.0	0.0/10.0	(Grade point, L, C and L.)
129	732.3	736.50			⊼728.22	⊙737.58	-4.1/14.2	-4.2	-4.2/14.3	(3-level section in fill.)
+60	723.9	736.50		4.3	-8.3	B.S. 10.09 ⊼747.67	-12.6/26.9	-12.6	-12.6/26.9	(Level section in fill.)
+34						F.S. 5.48				N. end stringer, Br. No. 18
N.D.+29	720.5	736.50		7.7	-8.3	742.19 Check on	-16.0/32.0	-16.0	-16.0/32.0	(Head of dump.)
T.D.+05	720.5	736.50		7.7	-8.3	B.M. No. 12	0.0/8.0	(-16.0)	0.0/8.0	(Toe of dump.)
128	720.1	736.50		8.1		(below.)				Bridge No. 18 {128+34
+90	712.2									6, 14' spans {127+50
T.D. L +81.4					-8.3	⊼737.23 F.S. 10.15	0.0/8.0 (-17.6)	←		(Toe of dump {left.)
T.D. R +73.8					-8.3	⊙727.08 B.S. 1.14		→	(-12.4) 0.0/8.0	right.)
N.D.+55	723.5	736.50		4.7	-8.3	⊼728.22	-15.0/30.5	-13.0	-10.4/23.6	(Head of dump.
+50										S. end stringer, Bridge 18.
+13					⊼737.23					Ditch 2.4×4.7×53.
127	727.8	736.50		9.4	0.7	⊼749.51 F.S. 12.58	-10.4/23.6	-8.7	-6.2/17.3	(3-level section in fill.)
+62	731.9	736.50		5.3	0.7	⊙736.93	-6.0/17.0	-4.6	0.0/10.0	(Grade point right.)
+37	736.5	736.50		(0.7)	(0.7)	B.S. 0.30	-2.0/11.0	0.0	+2.0/13.0	(Grade point center.)
+18	739.3	736.50		10.2	⊼749.51 13.0	⊼737.23	0.0/10.0	+2.8	+3.6/15.4	(Grade point left.)
126	741.4	736.50		8.1	13.0		+2.3/13.4	+4.9	+6.8/20.2	(3-level section in cut.)
+80	741.7	736.50		7.8	13.0		+5.2/17.8	+5.2	+5.2/17.8	(Level section in cut.)
+54	742.2	736.50		7.3	13.0	B.M. No. 12. 742.17	+10.0/25.0 +6.8/12.0	+5.7	+4.0/16.0	(4-level section in cut.)
125	746.1	736.50		3.4	13.0	B.S. 7.34 ⊼749.51	+12.8/29.2 +7.1/16.4	+9.6	+4.9/6.3 +6.4/19.6	(5-level section in cut.)
										Cuts, 20'; $1\frac{1}{2}$:1.
										Fills, 16'; $1\frac{1}{2}$:1.






CROSS-SECTION AT STATION 127+55
HEAD OF DUMP.
Grade Rod
15.0
8.3
13.0
10.4
4.7
30.5
23.6
130
+40
+21
Grade Point.
CROSS-SECTION AT STATION 127.
Grade Rod = 0.7
129
10.4
6.2
23.6
17.3
+60
Grade Point.
+29
H.D.
STATION 126+62
6.0
17.0
128
7.0.
Ravine.
STATION 126+37
Grade Point.
2.0
13.0
2.0
11.0
+55
H.D.
STATION 126+18
Grade Point.
15.4
127
Ditch.
+62
Grade Points.
CROSS-SECTION AT STATION 126
+37
+18
13.0
8.1
4.9
6.8
13.4
20.2
126
+80
+54
CROSS-SECTION AT STATION 125.
Grade Rod
12.8
13.0
3.4
9.6
4.9
7.1
6.4
16.4
6.3
29.2
19.6
125

ground. As shown in the form, the left hand page should be used for data and the other for the cross-section notes.

The organization and equipment of the cross-sectioning party when using the engineers' level is: (1) recorder (note book), (2) leveler (engineer's level), (3) rodman (self-reading leveling rod, 50-foot tape), (4) axeman (axe, sack of flat stakes, marking keel). The usual routine is: (1) Determine height of instrument by back sight on identified bench or turning point. (When a bench mark is remote and an original turning point can not be found, it may suffice in an emergency to check on the ground at several stations to the nearest 0.1 foot and use the mean height of instrument. Such places should be verified later.) (2) Having the height of instrument, check the original elevation of the station about to be cross-sectioned, reading the rod and checking off the elevation if it does not differ more than 0.1 foot or so; in case of a new plus, take a rod reading and record the elevation. (3) Determine the "grade rod" for the station by subtracting the height of instrument from the grade elevation; then note that cut or fill at any point of the cross-section is equal to surface rod minus grade rod; (counting rods as minus when downward from the plane of the level and those upward as plus, this rule gives results always plus for cut and minus for fill, which agrees with the conception that cross-section notes are rectangular coordinates of the sectional area referred to the center of the finished roadbed as an origin.) (4) If the ground is level transversely, that is, does not vary more than 0.1 foot or so within the limits of the proposed grading, then the distance from the center out to each side slope stake is half width of roadbed plus center cut or fill times rate of side slope; (thus for 20-foot roadbed, side slopes 1 to 1, and a cut of 18.6 feet, the distance out to slope stake on a level section would be 28.6 feet, or with a slope of 1½ to 1, the distance out would be 10 plus 1½ times 18.6, or 37.9 feet. Calculations of this sort should be done mentally in an instant.) (5) On three-level ground estimate the rise or fall of the surface from the center to about where the side slope stake should come, and add the same to, or subtract it from the center cut or fill, as the case may be; compute the distance out to the point where the side slope line would pierce the ground surface and test the same with tape, rod and level by the foregoing rule for cut or fill; continue to construct points on the side slope line until the common point is found. (6) The axeman marks "S. S." (slope stake) on one side of the stake with the cut or fill to the nearest 0.1 foot (as C 6.8 or F 10.2) and the station number on the other side; the stake is driven slanting towards or away from the center line according as it is cut or fill. (7) On five-level ground or, in general, on ground involving any number of points or angles in the section, the cut or fill is taken at each break. (8) Should there appear to be danger of land slips, the cross-sectioning should be carried well beyond the limits of the slope stake points. (9) The cross-section notes are recorded as in the accompanying form, expressing the coordinates of each point in the form of a fraction, and distinguishing the slope stake points by enclosure in a circle. (10) Having completed the cross-sectioning at the station, the same program is followed at the next point, first checking the elevation obtained in the original location levels; the grade rod should be determined as before by subtracting the height of instrument from the grade elevation, and then checked by applying to the preceding grade rod the rise or fall of grade from the preceding point. (11) Cross-sections should be taken as a general rule at every station and at such intermediate points as will insure a reliable measurement of the earthwork quantities. It is not necessarily the lowest and highest points that are required, but those points which, when joined by straight lines, will give the contents as nearly as possible equal to the true volume; if the "average end areas" method is to be used in calculating the quantities, sections should be taken every 50 feet when the difference of center height is as much as 5 feet; as a rule, slope stakes need not be set at cross-sections taken between stations. (12) "Grade point" stakes (marked 0.0), should be set where the center line and each edge of the roadbed pierce the ground; and also in side-hill sections in both cut and fill, where the roadbed plane cuts the ground line; if the width of roadbed is different in cut and fill, the greater half-width is commonly used in locating the side grade point; in the simplest case a contour line is perpendicular to the center line and the three grade points are at the same cross-section, forming two wedges; in the more usual case the contour line is diagonal, and the three grade points are not in the same section, so that two pyramids are formed; if the station numbers of the two side grade points differ by only a few feet, it is usual to simplify the record by taking the notes as for a wedge at the station number of the center grade point, although the side grade point stakes are set in their true positions; as a rule, a complete cross-section is taken at each grade point. (13) In cross-sectioning for the end of an embankment at a wooden trestle the end slope is made the same as the side slope, and the end and side planes are joined by conical quadrants; the distance between "heads of dump" (H. D.) is usually 10 feet (5 feet at each end) less than the total length of stringers; a complete cross-section is taken at the "head of dump," and the "toe of dump" (T. D.) on each edge of the end slope is located and recorded; on level ground the volume of the wedge-like solid so formed is found by dividing it into a triangular prism and two right conical quadrants; on ground sloping transversely the end of dump is made up of a middle prismoid and two conical quadrants, each of the latter being generated by a variable triangle revolved about a vertical axis through a corner of the top roadbed plane at "head of dump."

The calculations in the foregoing method of cross-sectioning may be simplified by preparing a table of distances out for the standard roadbed widths and slopes, or by using a special tape having the zero graduation at a distance from the end equal to the half-width of roadbed, and the remaining graduations modified to suit the side slope ratio. The calculations may be further simplified by using a special rod havig an endless sliding tape graduation. The student will be given practice with these labor saving devices after he has first acquired familiarity with the principles of cross-sectioning without these aids.

Cross-sectioning with rods alone is done in much the same manner as that described above. Two rods are used. The usual length of the rods is ten feet, and each is graduated to tenths and has a bubble vial in one or both ends. The slope stake point is determined by leveling out from the ground at the center stake with reference to the center cut or fill, each rod being held alternately level and plumb. Other points in the cross-section, as well as grade points, etc., are determined in the same manner. The notes are kept as in the other method. On very rough ground, the rod method is usually the more rapid. Some engineers cross-section on rough ground by taking the elevation of each point and plotting the notes on cross-section paper, then using the planimeter to determine the areas. Borrow pits are often cross-sectioned by taking elevations at the intersections of two series of parallel lines forming squares.

Land-Line Party.—It is the duty of the right of way party to secure data for the preparation of right of way deeds. The party should consist of at least four: (1) recorder, (2) transitman, (3) head chainman, (4) rear chainman, (the chainmen also to serve as axemen and flagmen as required.) Their equipment is the usual one of a transit party for such work. The party should secure ties with all section and other land lines whenever crossed. The notes should show station numbers and angles of intersection and distance along land line to the nearest identified land corner and also to important fences. As a rule, make the intersection by running through from one corner to the other. Where the line passes through a town, tie the center line to the plats, block lines, monuments, etc. Secure any records and make tracings of any plats, etc., at the recorder's office, that may be of service in preparing deeds.

Bridge and Masonry Party.—The bridge and masonry survey party will determine drainage areas for culverts and other waterways, prospect for foundations, and stake out trestles, masonry work, etc. The usual organization will be four men: (1) recorder (in charge), (2) transitman or leveler, (3) chainman, rodman, flagman, etc., (4) chainman, axeman, flagman, etc., as the work assigned may demand.

Resurvey Party.—The resurvey party will be assigned to such duties as the resurvey of yards, the collection of data for crossing frogs, running centers on old track, including spiraling, etc. It will usually be a party of four.

Seminary Work.—The purpose of the seminary work is: (1) to give the student a knowledge of the literature of railway engineering, and (2) to afford training in the collection and preservation of engineering information and data, and in the preparation of abstracts and reports of a technical nature. The reading will be done in accordance with a systematic outline and the notes will be submitted in prescribed form.

PROBLEMS IN RAILROAD SURVEYING.

PROBLEM G1. ADJUSTMENTS OF LEVEL AND TRANSIT.

(a) *Equipment.*—Engineers' level and transit, adjusting pin.

(b) *Problem.*—Test the essential adjustments of the assigned instruments and correct any discrepancies found.

(c) *Methods.*—This problem is designed to freshen the student's knowledge of the adjustments of the instruments, as well as to place the equipment in condition for accurate work. The adjustments will be made under the personal direction of the instructor. The student should attempt to be speedy as well as accurate in testing and making the adjustments.

PROBLEM G2. USE OF FIELD EQUIPMENT.

(a) *Equipment.*—Complete equipment for railroad transit and level party, as specified in foregoing pages.

(b) *Problem.*—Practice the detailed duties of each position in the transit and level party.

(c) *Methods.*—This problem is designed as a "breaking in" exercise preparatory to engaging in the regular field work of railroad location. With the manual in hand the duties of each position will be studied and practiced in turn.

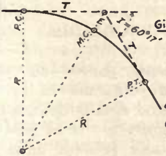
For example, each student will go through the following exercise with the transit as briskly as possible: (1) set transit over tack in hub, (2) level up, (3) set plate to zero, (4) reverse telescope and sight on back flag, (5) release needle, (6) plunge telescope, (7) read and record needle on back line prolonged, (8) sight at front flag pole, (9) read and record deflection angle right or left, (10) read and record needle on front line, (11) lift needle, (12) plunge telescope and check on back flag, (13) calculate needle angle and compare with plate reading, and if checked, shoulder transit; now repeat entire process at the same hub, more briskly than at first, if practicable, avoiding reference to preceding record until the full series of steps is completed.

Let the student prepare a similar numbered program for each of the other positions and practice the same systematically. This series of exercises may profitably occupy two or more assignments, since the speed and quality of the actual surveys to follow are certain to be much enhanced.

PROBLEM G3. PRELIMINARY FIELD CURVE PRACTICE.

(a) *Equipment.*—Transit party equipment, as prescribed in instructions.

Problem 2. Calculation of Curve Elements.



Given: $I=60°17'$ R. ; $D=4°17'$; $R(4°17')=1337.65$

Required: L, T and E. (a) By trigo. (b) By Table 1°C. (Results to 0.01 ft.)

$\frac{1}{2}I = \frac{60°17'}{2} = 30°08'.5$

tan $30°08'.5 = 0.58066$
exsec $30°08'.5 = 0.15636$

$60°17' = 60°.2833+$
$4°17' = 4°.2833+$

Part	Method (a)	Method (b)	Diff.
L	14.07^{39}	14.07^{39}	
T	776.71	776.77	0.06
E	209.15	209.17	0.02

Indicated Work.	Calculations.
Length of Curve, L. $L=\frac{60°17'}{4°17'}$ (a) $=\frac{3617'}{257'}=14.07^{39}$ (b) $=\frac{60°.2833}{4°.2833}=14.07^{39}$	257)3617(14.0739 o.k. 257 1047 1028 1900 1799 1010 771 2390 2313 77 60.2833)4.28333 428333 14.0739 174500 o.k. 171333 3167 2998 169 128 41 38
Tangent Distance, T. (a) $T=R\tan\frac{1}{2}I$ $=1337.65\times0.58066$ $=776.71$ (b) $T=\frac{T_{1°c.}}{D}$ $=\frac{3327.15}{4.2833}$ $=776.77$	1337.65 66085.0 66882 10701 80 8 776.71 o.k. $T_1(60°16') = 3326.0$ $T_1(60°18') = 3328.3$ $T_1(60°17') = 3327.15$)4.2833 (3) 2998.33 776.77 32882 o.k. 29983 2899 2570 329 300 29 30 776.77 776.71 0.06 Diff. due to approx. basis of method (b).
External Distance, E. (a) $E=R\,\text{exsec}\,\frac{1}{2}I$ $=1337.65\times0.15636$ $=209.15$ (b) $E=\frac{E_{1°c.}}{D}$ $=\frac{895.95}{4.2833}$ $=209.17$	1337.65 63651.0 13376 6688 803 40 8 209.15 o.k. $E_1(60°16') = 895.4$ $E_1(60°18') = 896.5$ $E_1(60°17') = 895.95$)4.2833 (3) 85667 209.17 3928 o.k. 3855 73 43 30 30 209.17 209.15 0.02 Diff. due to approx. basis of method (b).

(b) *Problem.*—Run out the assigned practice curves in the field, with the prescribed organization and conditions.

(c) *Methods.*—The preliminary curve practice is designed to give the student a practical knowledge of the principles of railroad curves and the routine methods used in location surveys. The several positions in the field party will be filled in succession, and each student is expected to respond heartily to the spirit of the practice, whatever his assigned duties. Each member of the party should engage in the calculations as far as practicable. The report of the field work should state the precision of linear and angular checks. The field practice will be based in part on the indoor curve problems.

PROBLEM G4. CURVE PROBLEMS.

(a) *Equipment.*—Drafting instruments, paper, etc.

(b) *Problem.*—Solve the assigned problems in railroad curves and submit results in a neat and draftsmanlike form.

(c) *Methods.*—(1) Draw a plain figure to the largest convenient scale. (2) State problem and present data in a concise and systematic manner. (3) show the separate steps clearly; first state formulas in general terms, then substitute values and give results; as a rule, show actual calculations adjacent to the indicated work; habitually verify results by an independent process; use common sense checks and contracted methods of calculation; in general, make full use of the opportunity to gain skill as a computer. (As a rule, the nearest 0.1 foot only is required in field measurements on curve location, but it is excellent practice, especially for the beginner, to preserve the nearest 0.01 foot in the calculations.)

CHAPTER IX.

ERRORS OF SURVEYING.

Errors.—Errors of observations are of three kinds, viz., (1) mistakes; (2) systematic errors; (3) accidental errors. Systematic errors includes all errors for which corrections can be made, as erroneous length of standard, errors of adjustment, refraction, etc. Accidental errors are those which still remain after mistakes and systematic errors have been eliminated from the results.

It has been found from experience that accidental errors are not distributed at random but follow mathematical laws. These laws are fundamental in the Theory of Least Squares and are: (1) small errors are more frequent than large ones; (2) positive and negative errors are equally numerous; (3) very large errors do not occur.

Arithmetical Mean.—The most probable value of a quantity obtained by direct measurements is the arithmetical mean of all the determinations where the observations are of equal weight, or is the weighted mean where the observations are of unequal weight.

Precision of Observations.—In the adjustment of observations it is often necessary to combine results of different degrees of precision or weight. It is also desirable to have some means of comparing observations so that the computer may know what degree of confidence to place in the results. The quantity commonly used for comparing the precision of observations is the probable error.

Probable Error.—The probable error is such a quantity that it is an even wager that the number of errors greater is the same as the number of errors less than the probable error. It is also the limit within which the probability is one-half that the truth will fall. For example, if 4.63 ± 0.12 is the mean of a number of observations, the true value is as likely to be between 4.51 and 4.75 as it is to be some value greater or less.

Probable error is also useful in finding the relative weights that should be given different sets of observations, as it has been found that the weights of observations vary inversely as the squares of their probable errors.

Formulas:

Let E_1 = probable error of a single observation.

E_m = probable error of the mean of all the observations.

n = the number of observations.

d = the difference between any observation and the mean of all the observations.

Σ = symbol signifying sum of.

Then from the Theory of Least Squares

$$E_1 = 0.6745 \sqrt{\frac{\Sigma d^2}{n-1}} \tag{1}$$

$$E_m = 0.6745 \sqrt{\frac{\Sigma d^2}{n(n-1)}} \tag{2}$$

$$= \frac{E_1}{\sqrt{n}} \tag{3}$$

The probable error of the weighted or general mean is

$$E_o = 0.6745 \sqrt{\frac{\Sigma p d^2}{(n-1)\Sigma p}} \tag{4}$$

where Σp = summation of the weights.

The probable error of a quantity with a weight p is equal to E_0 divided by the square root of p.

The probable error of Z where $Z = z_1 \pm z_2$ and R_1, r_1, and r_2 are the probable errors of Z, z_1 and z_2, respectively, is

$$R_1^2 = r_1^2 + r_2^2 \tag{5}$$

The probable error of Z, where $Z = az$ is

$$R_1^2 = a^2 r^2 \tag{6}$$

The probable error of Z, where $Z = z_1 z_2$ is

$$R_1^2 = z_1^2 r_2^2 + z_2^2 r_1^2 \tag{7}$$

This would be the probable error of the area of a rectangle where r_1 and r_2 are the probable errors of the sides z_1 and z_2, respectively.

Example.—As an example of the application of these formulas consider the two following series of measurements of an angle given in Table I. The first set was taken with a transit reading to 10 seconds, the second with a transit reading to 30 seconds.

FIRST TRANSIT.						SECOND TRANSIT.					
No.	Angle.			d	d²	No.	Angle.			d	d²
	°	′	″				°	′	″		
1	34	55	35	2	4	1	34	56	15	39	1521
2			35	2	4	2		55	30	6	36
3			20	13	169	3		54	30	66	4356
4			05	28	784	4		55	15	21	441
5		56	15	42	1764	5		56	00	24	576
6		55	40	7	49	6		55	45	9	81
7			10	23	529	7		55	30	6	36
8			30	3	9	8		55	30	6	36
9			50	17	289	9		56	00	24	576
10			30	3	9	10		55	45	9	81
Mean 34° 55′ 33″				$\Sigma d^2 = 3610$		Mean 34° 55′ 36″				$\Sigma d^2 = 7740$	
$E_m = 0.6745\sqrt{\frac{3610}{9\times10}} = \pm 4''.3$						$E_m = 0.6745\sqrt{\frac{7740}{9\times10}} = \pm 6''.3$					

The weights of these mean values vary inversely as the squares of the probable errors, or in this case the weights are as $\frac{1}{4.3^2}$ to $\frac{1}{6.3^2}$ or as 12 to 5. The most probable value of the angle measured with the two transits will be the weighted mean

$$Z = 34^\circ\ 55' + \frac{33 \times 12 + 36 \times 5''}{17}$$
$$= 34^\circ\ 55'\ 33''.9$$

The probable error of this result from (5) since

$$Z = \frac{12}{17} z_1 + \frac{5}{17} z_2 \quad \text{is}$$

$$R_1{}^2 = \left(\frac{12}{17}\right)^2 r_1{}^2 + \left(\frac{5}{17}\right)^2 r_2$$

Substituting $r_2^2 = \frac{12}{5} r_1^2$ we have

$$R_1^2 = \left(\frac{12}{17}\right)^2 r_1^2 + \left(\frac{5}{17}\right)^2 \frac{12}{5} r_1^2$$

$$= \frac{12}{17} r_1^2$$

$$R_1 = \pm 4.''3 \sqrt{\frac{12}{17}} = \pm 3''.6.$$

For other examples in the use of probable error see probable error of measuring a base line, probable error of setting a level target, probable error of setting a flag pole.

Angle Measurement.—The measurement of an angle requires two pointings and two readings. If r_r and r_s are the probable errors of reading and pointing, respectively; the probable error of the measurement of an angle will from (5) be

$$R_1 = \sqrt{r_r^2 + r_s^2}$$

If r_1 is the probable error of a single reading

$$r_r = r_1 \sqrt{2}$$

If the value of an angle is determined by n separate measurements the probable error due to reading will be

$$r_r = \frac{r_1 \sqrt{2}}{\sqrt{n}}$$

If the value of an angle is determined by measuring the angle n times by repetition the probable error due to reading will be

$$r_r = \frac{r_1 \sqrt{2}}{n}$$

It will thus be seen that the probable error due to reading is very much reduced by measuring an angle by the method of repetition. The errors of pointing, etc., however, make it doubtful whether it is ever advantageous to make n exceed 5 or 6 with an engineers' transit.

Angle Adjustment.—When the three angles of a triangle have been measured with equal care they should be adjusted by applying one-third of the error as a correction to each angle.

When the interior angles of a polygon having n sides have been measured with equal care they should be adjusted by applying one-nth of the error as a correction to each angle.

When n—1 angles and their sum angle at a point have been measured with equal care they should be adjusted by applying one-nth part of the error as a correction to each angle.

In a quadrilateral the true values of the angles fulfil the following geometrical conditions: (1) the sum of the angles of each triangle is equal to 180° plus the spherical excess (the spherical excess in seconds of arc is equal approximately to the area in square miles divided by 78); (2) the computed length of any side when obtained from any other side through two independent sets of triangles is the same in both cases.

When the angles of a quadrilateral have been measured, errors are certain to be present and the corrections that satisfy one of these conditions will not satisfy the other. The most probable values of the corrections to the angles are then determined by the Theory of Least Squares.

TESTS OF PRECISION.

Practical Tests.—In careful surveying where blunders are eliminated and the systematic and accidental errors are small and under control, it is found that the magnitude of the errors increases in close accord with the foregoing rational basis, that is, as the square root of the number of observations. The following practical tests of precision are based on this truth. (The diagrams have been prepared with a view to supply extra copies for insertion in the field note book where they may be consulted as the results are obtained.)

Linear Errors.—Cumulative or systematic errors usually increase directly as the length of the line chained, while compensating or accidental errors vary about as the square root of the length. While both kinds of errors affect all linear measurements, the former chiefly control the results of crude and the latter of accurate chaining. It is thus fairly consistent to express the precision of chaining in crude work in terms of the simple ratio of the length; but as the chaining becomes more and more exact, the variation of the differences between duplicate measurements approximates more and more closely to the law of square roots.

Coefficients of precision derived from the latter relation may be based on either 100-foot units or foot units in the distance chained, as preferred. The former basis is used in the chaining diagram, while the latter is found in the last paragraph of the explanatory matter on the second page referring to the precision of traverse surveys.

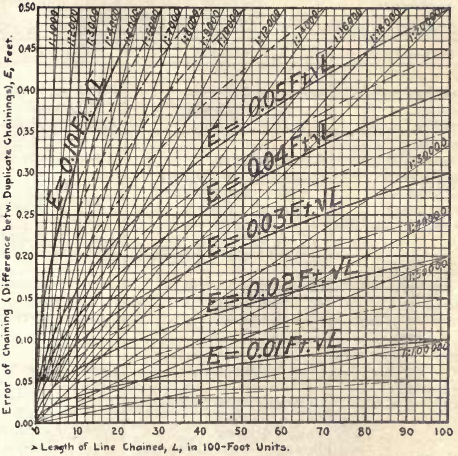
The diagram of chaining errors shows chaining ratios by right lines radiating from the origin, and the law of square roots by means of parabolas. The coefficient of precision for a given observed difference between duplicate chainings is determined by inspection from the diagram, interpolating between curves if an additional decimal place is desired in the result. In actual practice a pair of careful chainmen may determine the coefficient corresponding to a given degree of care, and then use this value either in testing their duplicate results, or in estimating the probable uncertainty of the lengths chained.

For accurate chaining with the steel tape, duplicate measurements reduced for temperature, etc., or made under sensibly identical conditions, should not differ more than 0.05 foot into the square root of the distance in 100-foot units. Careful work with the common chain (estimating fractions to 0.01 foot) should not differ more than 0.1 foot into the square root of the distance in 100-foot units.

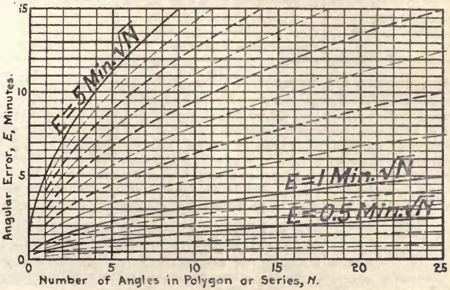
Angular Errors.—In measuring deflection angles by altitude reversals, as in railroad traversing, there is, of course, a cumulative discrepancy due to the collimation error; but generally speaking, careful angular measurements with good instiuments are subject only to compensating or accidental errors. Under the latter conditions the magnitude of the error of closure in a series of angles, either in a closed polygon or about a point, varies about as the square root of the number of angles. This relation is indicated graphically in the diagram of angular errors.

In measuring angles with a transit reading to the nearest minute, the compensating uncertainty of a single reading is probably somewhat under 0.5 minute per angle, or about one minute for the closure of a triangle. If a reading glass

THE PRECISION OF CHAINING.



THE PRECISION OF ANGULAR MEASUREMENTS.



THE PRECISION OF TRAVERSE SURVEYS.

The error of closure of a traverse is usually expressed as the ratio of the calculated linear error to the length of the perimeter of the field or polygon. The following table shows the limits prescribed by various authorities

Prescribed Limits For Closure Of Traverses.

Authority.	Conditions.	Limits.
Gillespie. (1855). "Surveying," p. 149.	*Compass Surveys.*	*1:300 to 1:1000*
Alsop. (1857). "Surveying," p. 199.	*Compass Surveys.* *Transit Surveys.*	*1:500* *1:1000 to 1:1500*
Davies. (1870). "Surveying," p. 127.	*Farm Surveys.*	*1:500 to 1:1000*
Jordan. (1877). "Handbuch der Vermessungs-Kunde," Vol. I, p. 296.	*German Gov't Surveys.* *Baden Instructions.* *Prussian Instructions.* *Swiss Gov't Surveys.* *Ordinary Country.* *Mountainous Country.*	 *1:400* *1:333 to 1:1000* *1:400 to 1:800* *1:267 to 1:533*
Hodgman. (1885). "Surveying," p. 119.	*Compass Surveys.*	*1:300 to 1:1000*
Johnson. (1886). "Surveying," p. 201.	*Farm Surveys.* *City Surveys.*	*1:300* *1:1000 to 1:5000*
*Baker. * (1888). "Engineers' Surveying Instruments," p. 53.*	*(See Footnote).*	*(See Footnote).*
Carhart. (1888). "Surveying," p. 161.	*Ordinary Farm Surveys.* *Level Ground.* *Rough Ground.* *Average Transit Surveys.*	*1:500* *1:1000* *1:200 to 1:300* *1:1200*
Wood. *(Roanoke, Va., 1892).* *(Baltimore, Md., 1894).*	*(See Footnote).* *Precise Traverses with Repeated Angles.*	*(See Footnote).* *1:10 000* *1:15 000 + .04 Ft.*
Raymond. (1896). "Surveying," p. 144.	*Ordinary Farm Surveys.* *Good Farm Surveys.*	*1:500* *1:2000*

**Baker derives the formula* $E = P\sqrt{\frac{1}{d^2} + \frac{a^2}{12\,000\,000}}$ *where E is the permissible linear error of closure, P the length of the perimeter, 1:d the ratio of the chaining error, and a the angular error of closure in minutes. A thorough test of this formula under a wide range of conditions proves it to be trustworthy.*

However, the use of a chaining ratio, 1:d, presumably of fixed value for the same chainmen, does not accord with the results of experience in careful work; for it is found that the differences between duplicate chainings vary about as the square root of the length of line.

On the following page a simplified formula is obtained by assuming the more consistent relation just stated for the chaining errors. The results are about the same as those obtained with Baker's formula, and the form of the expression is identical with that used by Wood in the Baltimore Survey.

THE PRECISION OF TRAVERSE SURVEYS.

The reasonable or permissible error of closure of a traverse survey may be determined by the formula derived below, provided the errors of field work are under control and their magnitude is known, at least approximately.

Let P = length of perimeter.
L = calculated error of latitudes.
D = calculated error of departures.
E_a = actual or calculated linear error of closure of traverse.
c = coefficient of precision of chaining.
C = linear error of closure due to chaining errors.
a = angular error of closure in minutes.
A = linear error of closure due to angular errors.
E_p = permissible or reasonable linear error of closure due to errors of chaining and angle.

In the triangle of error the hypothenuse is $E_a = \sqrt{L^2+D^2}$. In Diagram A below values of E_a may be read close enough for most cases. Diagram A may also serve as a crude graphical traverse table, and blunders in the field work may be located by it.

In careful chaining by men of some training, the error varies about as the square root of the distance. If c be the compensating error for the unit distance, then $C = c\sqrt{P}$.

The angular error of closure in careful surveys probably occurs among the sides in proportion to their lengths. Assuming this to be the case, the resulting linear error is $A = aP.\text{arc }1' = .0003aP$.

In good work the errors are small in amount and equally liable to be plus and minus. Hence, the probable error of closure due to the two causes, i.e. the reasonable or permissible linear error of closure is $E_p = \sqrt{A^2+C^2} = \sqrt{(.0003aP)^2+c^2P}$.

This formula may be much simplified by completing the square and dropping the negative term under the radical, whence with sufficient exactness, there results the general formula

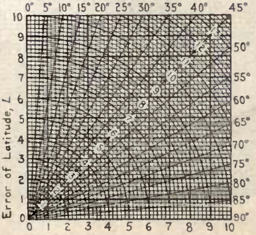
$$E_p = .0003\,aP + 1700\,c^2 \quad . \; . \; . \; . \; (1)$$

The very exact standard, $P \div 15000 + .04$ ft., used at Baltimore, (see table, preceding page), may be obtained from (1) by making a somewhat less than $\frac{1}{4}$ minute, and $c = .005$ ft., these values being consistent with the field work of that survey.

The value of c may be determined for the given chainmen, or the chaining term of (1) may be taken as follows:- For best work ($c < .005$ ft.), .05 ft.; for average work ($c < .010$ ft.), .2 ft.; for fair work ($c < .015$), .4 ft.; and for poor work ($c < .020$), .8 ft. In careful traverse surveys the angle term alone affords a rigid test, so that formula (2) may be used except when $a = 0$. Diagram B gives (2) for the general run of traverse problems.

$$E_p = .0003\,aP = \frac{3aP}{10000} \quad . \; . \; . \; . \; (2)$$

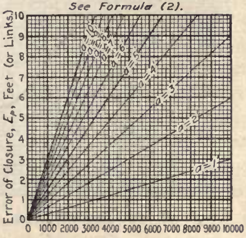
A. Actual Error.



Error of Departure, D.

B. Permissible Error.

See Formula (2).



Length of Perimeter, P, Feet (or Links.)

THE PRECISION OF LEVEL CIRCUITS.

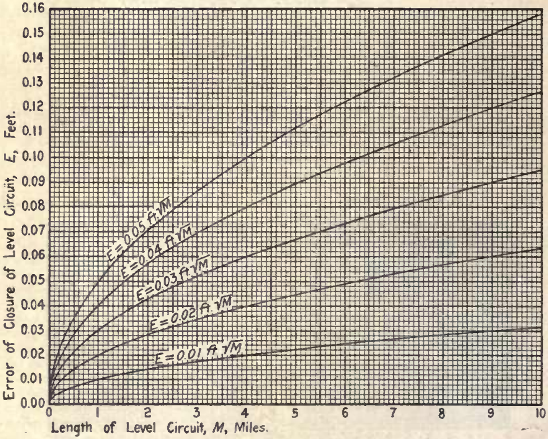
The precision of spirit leveling is expressed by the formula

Error of Closure = Constant $\sqrt{\text{Length of Circuit}}$.

In the following summary of practice in representative surveys of the United States, E is the maximum limit of error of closure of a level circuit having a length of K kilometers or M miles.

NAME OF SURVEY.	MAXIMUM PERMISSIBLE ERROR OF CLOSURE. Metric Units. Coefficient to nearest mm.	British Units. Coefficient to nearest 0.001 ft.	British Units. Coefficient to nearest 0.01 ft.
Chicago Sanitary District.	$E = 3\text{mm}\sqrt{K}$	$= 0.012 \text{ ft.}\sqrt{M}$	$= 0.01 \text{ ft.}\sqrt{M}$
Missouri River Commission.	$E = 3\text{mm}\sqrt{2K}$	$= 0.018 \text{ ft.}\sqrt{M}$	$= 0.02 \text{ ft.}\sqrt{M}$
Mississippi River Commission. (1891).	$E = 3\text{mm}\sqrt{2K}$	$= 0.018 \text{ ft.}\sqrt{M}$	
Mississippi River Com'n (Before 1891).	$E = 5\text{mm}\sqrt{K}$	$= 0.021 \text{ ft.}\sqrt{M}$	
United States Coast Survey.	$E = 5\text{mm}\sqrt{2K}$	$= 0.029 \text{ ft.}\sqrt{M}$	$= 0.03 \text{ ft.}\sqrt{M}$
United States Lake Survey.	$E = 10\text{mm}\sqrt{K}$	$= 0.042 \text{ ft.}\sqrt{M}$	$= 0.04 \text{ ft.}\sqrt{M}$
United States Geological Survey.	$E =$	$0.050 \text{ ft.}\sqrt{M}$	$= 0.05 \text{ ft.}\sqrt{M}$

A simple practical test of the degree of precision attained in spirit leveling is found in the last column of the above table. This graduated scale of precision is given below graphically for distances to ten miles.



THE PRECISION OF LEVEL CIRCUITS.
(For Good Average Practice.)

When the length of the level circuit is known in 100-ft. stations, or when merely the number of settings of the instrument and the approximate average distance covered per setting are known, the following modifications of the preceding test are valuable.

Let E *= maximum permissible error of closure of level circuit.*
M *= length of level circuit in miles.*
L *= " " " " " 100-ft. stations.*
L' *= approximate average distance covered per setting of the instrument in 100-ft. stations.*
S *= number of instrumental settings in the circuit.*

For good average work with the engineers' level

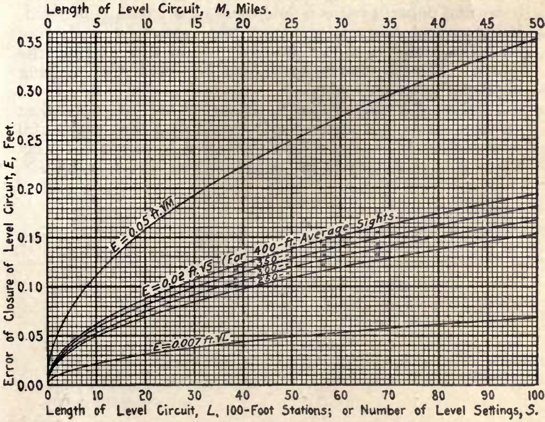
$$E = 0.05\text{ ft.}\sqrt{M}$$

from which $$E = 0.007\text{ ft.}\sqrt{L}$$

and $$E = 0.007\text{ ft.}\sqrt{L'S}$$

Substituting for 400-ft. average sights, $L' = 8$, $E = 0.0195\text{ ft.}\sqrt{S}$
" 350-" " " $L' = 7$, $E = 0.0182\text{ ft.}\sqrt{S}$
" 300-" " " $L' = 6$, $E = 0.0169\text{ ft.}\sqrt{S}$
" 250-" " " $L' = 5$, $E = 0.0154\text{ ft.}\sqrt{S}$

For a very rapid approximate check under ordinary conditions, it may be assumed that $E = 0.02\text{ ft.}\sqrt{S}$. *A graphical representation of these formulas is given below.*



be used and the vernier reads to the nearest half minute, the uncertainty is still further reduced.

Again, in estimating the needle reading of a compass to the nearest 5 minutes (one-sixth part of half-degree), the uncertainty of reading alone is perhaps 3 minutes, although this is increased by other conditions such as sluggishness of needle, etc., probably causing an uncertainty of as much as 5 minutes per angle, which latter limit would produce an error of closure of a triangle of say 10 minutes, and of a five-sided polygon of perhaps the same amount. (See diagram.)

Traversing Errors.—The errors of traversing are made up of the combined errors of linear and angular measurements. If the error of closure as determined from the latitudes and departures is large, the work should be scanned closely to detect blunders such as the substitution of sine for cosine, errors of 100 feet in chaining, misplacing decimal point, etc. After establishing the consistency of the residual errors, they should be distributed either in proportion to the lengths of the several courses, as in the more common usage, or in the proportion of the respective latitudes and departures, as would seem to be more consistent. If the several courses have not been surveyed with like precision, weights should be assigned in distributing the errors. Absurd refinements should be avoided in making the distribution of errors.

Leveling Errors.—Perhaps in no phase of surveying measurements is it more clearly established that accidental errors follow the law of square roots than in careful leveling. The precision diagrams are based on best current usage.

CHAPTER X.

METHODS OF COMPUTING.

Introduction.—To no one is the ability to make calculations accurately and rapidly of more value than to the engineer. Many fail to appreciate the value of rapid methods of calculation, and have no conception of the amount of time that can be saved by the skillful use of arithmetic, logarithms, reckoning tables and computing machines.

In the field the engineer has to depend upon the ordinary methods of arithmetic, or a table of logarithms for his results. The use of these aids should therefore receive special attention, for the engineer cannot afford to lose the time of his assistants while he makes unnecessary or extended computations.

In the office tables of squares, reckoning tables, slide rules and computing machines can be used in many cases with profit.

Consistent Accuracy.—It is safe to say that at least one-third of the time expended in making computations is wasted in trying to attain a higher degree of precision than the nature of the work requires.

In making arithmetical computations where decimals are involved it is a common practice to carry the result out to its farthest limit and then drop a few figures at random.

In using logarithms time and labor are lost by using tables that are more extensive than the data will warrant. The relative amount of work in using four, five, six and seven-place tables is about as 1, 2, 3 and 4. Besides the extra labor involved, the computer has a result that is liable to give him an erroneous idea of the accuracy of his work.

In making computations, in general, calculate the result to one more place than it is desired to retain.

If several numbers are multiplied or divided, a given percentage of error in any one of them will produce the same per cent of error in the result.

In taking the mean of a series of quantities it is consistent to retain one more place than is retained in the quantities themselves.

In direct multiplication or division retain four places of significant figures in every factor for an accuracy of about one per cent.; retain five places of significant figures in every factor for an accuracy of about one-tenth of one per cent.

LOGARITHMIC CALCULATIONS.

Logarithm Tables.—Logarithm tables contain the decimal part of the logarithm called the mantissa, the integral part called the characteristic is supplied by the computer.

Four-place tables give the mantissa to four decimal places of numbers from 1 to 999, and by interpolation give the mantissa of numbers from 1 to 9,999. Four-place logarithms should be used where four significant figures are sufficient, and should not be used where an accuracy greater than one-half of one per cent is required.

Five-place tables give the mantissa to five decimal places of numbers from 1 to 9,999, and by interpolation give the mantissa of numbers from 1 to 99,999. Five-place logarithms should be used where five significant figures are sufficient, and should not be used where an accuracy greater than one-twentieth of one per cent. is required. Five-place tables are sufficiently accurate for most engineering work.

Six-place tables give the mantissa to six decimal places of numbers from 1 to 9,999, and by interpolation give the mantissa of numbers from 1 to 99,999, the same as the five-place tables. Six-place tables are of no practical value as the labor of using a six instead of a five-place table is about as 2 to 3, and as the interpolation for the next significant figure is made with larger differences; it is less reliable than with the five-place table.

Seven-place tables give the mantissa to seven decimal places of numbers from 1 to 99,999, and by interpolation of numbers from 1 to 999,999. Seven-place tables are rarely needed in engineering work, except in triangulation work where the angles are measured by repetition.

ARITHMETICAL CALCULATIONS.

Requirements.—To become a rapid computer the following requirements are essential:

(1) A good memory for retaining certain standard numbers for reference.

(2) The power of performing the ordinary simple arithmetical operations of multiplication, division, etc., on numbers with facility, quickness and accuracy.

(3) The power of registration, *i. e.*, of keeping a string of numbers in the mind and working accurately upon them.

(4) The power of devising instantly the best method of performing a complicated problem as regards facility, quickness and certainty.

It is obvious that all do not have the ability to become rapid computors, but even these can become fairly skillful by constant practice and perseverance. The ordinary processes of arithmetic should be performed with numbers in all possible positions. No more figures should be put down than necessary, and all operations should be performed mentally whenever possible. In the mental part the results should alone be stated, much time being lost by repeating each separate figure.

Checks.—In order to check his work the computer should keep the following well known properties of numbers well fixed in his mind:

(1) The sum or difference of two even or of two odd numbers is even.

(2) The sum or difference of an even and odd number is odd.

(3) The product of two even numbers is even.

(4) The product of two odd numbers is odd.

(5) The product of an even number and an odd number is even.

(6) Checking results by the familiar operation of casting out the 9's depends upon the following properties of numbers:

(*a*) A number divided by 9 leaves the same remainder as the sum of the digits divided by 9. For example:

$$4384 \div 9 = 487 + 1$$
$$(4+3+8+4) \div 9 = 2+1$$

(*b*) The excess of 9's in the product equals the excess of 9's in the product of the excesses of the factors.

473,295	Excess = 3		
4,235	Excess = 5		
	15	Excess = 6	Check
2,004,404,325	Excess =	6	

(*c*) The excess of 9's in the dividend equals the excess of 9's in the product of the excesses in the divisor and quotient plus the excess in the remainder:

56)2443	Excess in divisor $=2$
43+35	Excess in quotient $=7$
	Excess in remainder $=8$
	Excess in $(2\times 7+8)=4$ } Check.
	Excess in dividend $=4$ }

(7) Results should be checked by taking aliquot parts wherever possible, and by performing the operations in inverse order or performing inverse operations. Computations performed by means of logarithms should be checked by making the computations roughly by means of arithmetic. The probability of error should be recognized and precaution taken to verify results.

ADDITION.—Since the eye is accustomed to pass from left to right time can be saved, where the columns are not too long, by adding in the same way. The device of increasing or diminishing the numbers to make them multiples of ten and then subtracting or adding to the result is very convenient, especially where several columns are added at one time.

Ex. 1.—

	96	
	47	143
212	69	
	32	
	87	331
	49	
	380	

The mental work in detail is as follows:

$100+47=147$; $147-4=143$; $143+70=213$; $213-1=212$; $212+30+90=332$; $332-1=331$; $331+50=381$; $381-1=380$

Expert accountants use the method of adding columns in groups of 10, 20, 30, etc., small figures, indicating the number of the group, being placed along the column at intervals depending upon the computer. This method is well adapted to the addition of long columns where one is liable to be called away from his work. The progress of the work being then shown by the number of the group plus the excess.

MULTIPLICATION.—In order to make the best use of the methods given, the computer should have perfect command of the multiplication table as far as 20 at least.

Multiples of 10.—To multiply by some number which is a factor of 10 or some multiple of 10, for example: Multiply A by B, where $B=\frac{C10^n}{d}$

Annex n ciphers to A, multiply by C and divide by d.

Ex. 1.—$4{,}324\times625=4{,}324(\frac{5\times10^3}{8})=(4{,}324{,}000\times5)\div8=2{,}702{,}500.$

Ex. 2.—$7{,}924\times25=792{,}400\div4=198{,}100.$

Squaring Small Numbers.—Numbers may be squared mentally by the following rule: Add to or subtract from one factor enough to make its units figure zero. Subtract from or add to the other factor the same amount. Multiply together this sum and difference, and to the product add the square of the amount by which the factors were increased or diminished.

Proof.— $a^2-b^2=(a+b)(a-b)$

$a^2=(a+b)(a-b)+b^2$

Ex. 1.— $(76)^2=(72\times80)+4^2=5{,}776$

Ex. 2.— $(127)^2=(124\times130)+3^2=16{,}129$

Ex. 3.— $(6\frac{1}{4})^2=(6\times6\frac{1}{2})+(\frac{1}{4})^2=39\frac{1}{16}$

Ex. 4.— $(6\frac{1}{2})^2=(6\times7)+(\frac{1}{2})^2=42\frac{1}{4}$

Ex. 5.— $(7.5)^2=(7\times8)+(5)^2=56.25$

It will be seen that the process is very simple where the units place is 5.

(2) When the tens differ by unity and the sum of the units equals 10, numbers may be multiplied by the following rule: From the squares of the tens of the larger number subtract the square of the units of the larger number. For the numbers may be represented by $(a+b)$ and $(a-b)$, and the product will be $(a+b)(a-b)=a^2-b^2$.

Ex. 6.—$(93\times87)=90^2-3^2=8{,}100-9=8{,}091.$

(3) The product of composite numbers is best obtained mentally by resolving them into their factors and taking the products of the factors.

Ex. 7.— $26\times36=9\times13\times8=936$

Ex. 8.— $48\times24=(24)^2\times2=1,152$

(4) Having the square of any number the square of the number next higher is obtained by the following rule: To the known square add the number and the next higher and the result will be the square of the next higher number.

Ex. 9.— $(25)^2=625.$ $(26)^2=625+25+26=676$

(5) A very close approximation to the square of a quantity which is very near unity is obtained by adding algebraically two times the difference between the quantity and unity to the quantity.

Proof. — $(1\pm b)^2=1\pm2b+b^2=1\pm2b$, (approximate).

Ex. 10.—$(1.05)^2=1+2\ (1.05-1)=1+.10=1.10$

Ex. 11.—$(.94)^2=1-2\ (1-.94)=1-.12=.88$

Ex. 12.—$(2.034)^2=2^2(1+2\times.017)=4(1.034)=4.136$

Cross-Multiplication.—This consists in aking the product of each digit in the multiplicand by each digit in the multiplier and taking the sums, products of the same denomination being determined thus: units×units gives units; tens×units and units×tens gives tens; units× hundreds, tens×tens and hundreds×units give hundreds, etc. All products are added mentally, only the final result being put down.

Ex. 1.—$(2,347)^2=5,508,409$ the final result being all that it is necessary to write down. The mental work is as follows, the figures in heavy type being figures in the product: $7\times7=\mathbf{49}$; $4+2\ (7\times4)=\mathbf{60}$; $6+2(7\times3)+4^2=\mathbf{64}$; $6+2\ (2\times7)+2\ (3\times4)=\mathbf{58}$; $5+2\ (2\times4)+3^2=\mathbf{30}$; $3+2(3\times2)=\mathbf{15}$; $1+2^2=\mathbf{5}$.

Ex. 2.—The product of any two numbers may be found in the same manner.

$$\begin{array}{r} 9{,}432 \\ 2{,}583 \\ \hline 24{,}362{,}856 \end{array}$$

The mental work is as follows: $3\times2=6$; $3\times3+8\times2=25$; $2+3\times4+8\times3+5\times2=48$; $4+3\times9+8\times4+5\times3+2\times2=82$; $8+8\times9+5\times4+2\times3=106$; $10+5\times9+2\times4=63$; $6+2\times9=24$.

Ex. 3.—The process of cross-multiplication may be simplified as follows: Required to multiply 4,328 by 736; write the multiplier on a slip of paper in inverse order and place it below the multiplicand with the left hand figure below the units place of the multiplicand thus:

$$\begin{array}{r} 4{,}328 \\ \boxed{637} \end{array}$$

Multiply together the figures in the same vertical column, $6\times8=48$; set down the 8 and carry the 4; then move the slip one space to the left, thus,

$$\begin{array}{r} 4{,}328 \\ \boxed{637} \\ \hline 8 \end{array}$$

Multiplying together the figures in the same vertical columns and taking the sum, $4+6\times2+3\times8=40$; set down the 0 and carry the 4; then move the slip one space to the left, multiplying together the figures in the same vertical columns, adding, etc., we will finally have the work standing thus:

$$\begin{array}{r} 4{,}328 \\ \boxed{637} \\ \hline 3{,}185{,}408 \end{array}$$

Removing the slip we have

$$\begin{array}{r} 4{,}328 \\ 736 \\ \hline 3{,}185{,}408 \end{array}$$

The multiplier may be written on the bottom of a sheet in inverse order and placed above the multiplicand instead as above described. The work, however, is very much simplified by simply writing the multiplier in inverse order without using the slip:

$$\begin{array}{r} 4{,}328 \\ 637 \\ \hline 3{,}185{,}408 \end{array}$$

The mental work being as follows: $6\times8=48$; $4+6\times2+3\times8=40$; $4+6\times3+2\times3+7\times8=84$; $8+6\times4+3\times3+7\times2=55$; $5+3\times4+7\times3=38$; $3+7\times4=31$. It will be seen that this device removes most of the mental strain, there being no cross-products.

CONTRACTED MULTIPLICATION. — In multiplying decimals, when the product is required to a few places of decimals. the work may be shortened as follows: Required a product correct to the nth decimal place. Write the multiplier with its figures in inverse order, its unit place under the nth decimal place of the multiplicand. Multiply the multiplicand by the figures in the multiplier, beginning with the ri ht hand figure: rejecting those figur s in the multiplicand which are to the right of the figure used as a multiplier, increasing each product by as many units as would have been carried from the rejected part of the multiplicand, taking the nearest unit in each case place the right hand figure of each partial product in the same column, and add as in common multiplication.

In most cases it is best to carry one more place than required. The following examples illustrate the process:

Ex. 1.—The radius of a circle is 420.17 ft. What is its semicircumference to nearest 0.01 ft.? ($\pi = 3.14159265$)

In the work below the partial products in the contracted multiplication are seen to correspond to the partials of the common method, taken in reverse order, the part to the right of the vertical line being rejected. The contracted multiplication is carried one more place than required. A dot is placed above each figure when it is rejected from the multiplicand.

```
  ......
  420.170
56295141.3
----------
 1260510
   42017
   16807
     420
     210
      38
       1
----------
 1320.003
```

```
      420.17
     3.141593
-----------------
       1|26051
      37|8153
     210|085
     420|17
   16806|8
   42017|
  126051|
-----------------
1320.003|13081
```

Ex. 2.—The observed length of a line is 2231.63 ft. with a tape having a length of 100.018 ft. Required the reduced length of the line to the nearest 0.01 ft.

Noting that each foot of the tape = 1.00018 ft.

```
 ......
 2231.63
 81000.1
---------
 223163
     22
     18
---------
 2232.03
```

```
         2231.63
         1.00018
-----------------
      17|85304
      22|3163
  223163|000
-----------------
 2232.03|16934
```

Ex. 3.—Same observed length with a tape 99.982 ft. long. Required the reduced length.

Each foot of the tape = 0.99982 (= 1—0.00018) ft.

```
  ......
  2231.63
 81000.0−
---------
      22
      18
  ------
   −0.40
---------
  2231.23
```

```
      2231.63
      0.99982
--------------
      446326
     1785304
    2008467
   2008467
  2008467
--------------
 2231.2283066
```

Ex. 4—To compare contracted multiplication with logarithmic work, calculate 861.3 ft. × sin 17°19′ to the nearest 0.1 ft.

```
  861.3
56792.0
-------
  1723
   776
    60
     5
-------
 256.4
```

$\log 861.3 = 2.935154$
$\log \sin 17°19' = 9.473710$
$\log (256.4) = 2.408864$

CONTRACTED DIVISION.—If the quotient is desired correct to the nth decimal place, the followitg method may be used: Find one-half of the desired figures in the quotient in the usual way and do not bring down a figure for the last remainder. Drop a figure from the right of the divisor and find another figure in the quotient. Then without bringing down any more figures continue to discard figures from the divisor until the required places are obtained.

Ex. 1.—Divide 443.9425 by 24.311 to nearest hundredth. There will be four figures in the quotient, so we will find the first two in the ordinary way. A dot is placed over each figure in the divisor when it is rejected.

```
24.32 ) 443.9425 ( 18.25
        2432
        -----
        20074
        19456
        -----
          618
          486
          ---
          132
          122
          ---
           10
```

Divisor Near Unity.—When the divisor is near unity a very close approximation is given by the method shown in the following problems:

Ex. 1.— $\frac{5}{1.003254} = 5\,(1-.003254) = 5 \times .996746 = 4.98373$ correct to within one unit in the fifth place.

Ex. 2.— $\frac{7}{.9982} = 7\,(1+(1-.9982)) = 7 \times 1.0018 = 7.0126$ correct to the last place.

CONTRACTED SQUARE ROOT. A result correct to a required number of decimal places may be found by a process similar to the method employed for contracted division.

Ex. 1.—Required the square root of 12,598.87325 correct to thousandths. We see by inspection that the root will contain six figures. Find in the ordinary way the first three figures. Form a new trial divisor in the usual way, and bring down only one figure for the dividend in place of two. Find the remaining figures by contracted division.

```
          .  .  .  .  .  .
          12598.87325(112.245
          1
       21)25
          21
      222)498
          444
      224)548
          448
          100
           89
           11
           11
            0
```

The last figure brought down is not increased whatever it may be followed by, since the contracted process tends to make the result a little too large. This method may be applied to the extraction of cube roots, where it saves much work in finding long trial divisors.

Square Root of Small Numbers.—The approximate square roots of small numbers may be found by means of the following rule: Divide the given number by the number whose square is nearest the given number. The arithmetical mean of the quotient and divisor will be the approximate square root of the number. The nearer the number is to a perfect square the less the error. For example,

Ex. 1.—$\sqrt{35} = (\frac{35}{6} + 6) \div 2 = 5.92.$

Ex. 2.—$\sqrt{8} = (\frac{8}{3} + 3) \div 2 = 2.83.$

Ex. 3.—$\sqrt{79} = (\frac{79}{9} + 9) \div 2 = 8.89.$

Ex. 4.—$\sqrt{128} = (\frac{128}{11} + 11) \div 2 = 11.31.$

Square Root by Subtraction.—While it possesses no points of merit in this connection, it would not be proper to pass the subject of square root without presenting the novel method of extracting square roots used with the Thomas Computing machine. The method depends upon the relation existing between odd numbers and squares in the system of numbers having a radix ten. If we sum up the odd numbers, beginning at 1, we will observe the following relation:

$$1=1^2;\ 1+3=4=2^2;\ 1+3+5=9=3^2;\ 1+3+5+7=15=4^2,$$

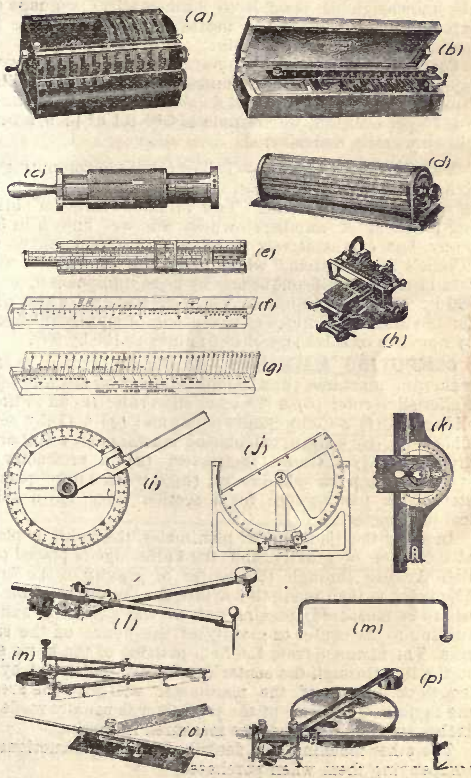
etc. It will be seen that the square root of the sum in each case is the number of the group.

The method of extracting square roots is as follows: Point off in periods of two figures each: subtract from the left hand period the odd numbers in order, beginning at unity, until a remainder is obtained less than the next odd number. Write for the first figure in the root the number which represents the number of subtractions made. Double the root already found and annex unity. Subtract as before, using for subtrahends the successive odd numbers, the root figure being the number of subtractions made.

Ex. 1.—Extract the square root of 53,824.

```
    .  .  .
    53824(232
    1
    4
    3 ....... 2 subtractions.
41)138
    41
    97
    43
    54
    45 ..... 3 subtractions.
461)924
    461
    463
    463... 2 subtractions.
      0
```

MISCELLANEOUS FORMULAS.—The engineer should have ready knowledge concerning approximate formulas and values. This knowledge can be obtained by the expenditure of very little energy and time if rightly applied. For ready


(a)
(b)
(c)
(d)
(e)
(f)
(g)
(h)
(i)
(j)
(k)
(l)
(m)
(n)
(o)
(p)

computation and reference he should reduce as much of his knowledge as possible to mathematical language and express known relations by means of formulas. The following will illustrate this point.

Cost of Sewer Pipe.—The Western Price List of sewer pipe is comprised in the formula, $C=0.4\ d^2+14$. Where C=cost in cents per foot and d=diameter of pipe in inches For 75 per cent off, the formula is $C^1=0.1\ d^2+3.5$, a formula very easily remembered.

RECKONING TABLES.—Tables for use in computing are so numerous and well known that it would be useless to try to refer to them by name. Two valuable tables for obtaining products of numbers—which are well known in Germany, but comparatively unknown in this country—are, "Crelle's Rechentafeln," which gives the products of numbers of three significant figures by three significant figures to 999 by 999; and "Zimmerman's Rechentafeln," which gives the products of numbers of two places of signigcant figures by numbers of three significant figures to 100 by 999.

COMPUTING MACHINES.—In Fig.40, (a) is a Kuttner reckoning machine; (b) a Thomas computing machine; (c) a Fuller slide rule; (d) a Thacher slide rule; (e) an ordinary slide rule; (f) a Colby Stadia slide rule; (g) a Colby sewer slide rule; (h) a Grant calculating machine; (i) a full circle protractor; (j) a Crozet protractor; (k) a protractor tee square; (l) a polar planimeter; (m) a "jack knife" planimeter; (n) a pantagraph; (o) a section liner; (p) a spherical planimeter.

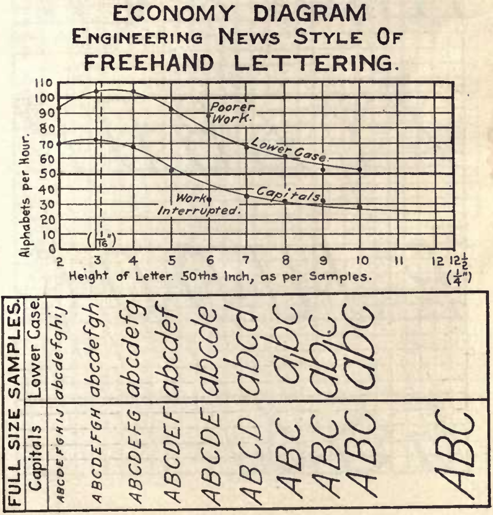
In using the "jack knife" planimeter, the point is placed at the center of gravity, and the knife edge is placed on a line passing through the center of gravity of the figure. The point is then made to traverse the perimeter of the figure to be measured; passing out to the perimeter and returning to the center of gravity of the figure on the same line. The distance from the final position of the knife edge to the line through the center of gravity, multiplied by the length of the arm of the planimeter will give the area of the figure. The arm of the protractor is usually made ten inches long and the distance measured in inches.

The other machines are described in the instructions accompanying them when purchased.

CHAPTER XI.

FREEHAND LETTERING.

Practice Plates.—A magnified scale is used in the first six plates to give familiarity with form of letter and numeral, and also to produce freedom of hand motion. The six plates should first be made with a soft pencil sharpened to a needle point, and afterwards with pen and india ink. In Plate 7 the height of letter is that prescribed in Chapter I. This standard size is not only well adapted to field notes and general drafting, but is economical of execution, as shown by the diagram.



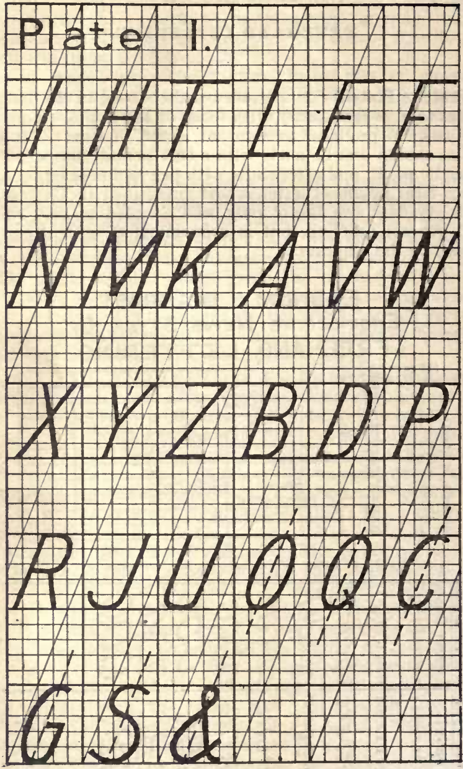


Plate I.
I H T L F E
N M K A V W
X Y Z B D P
R J U O Q C
G S &

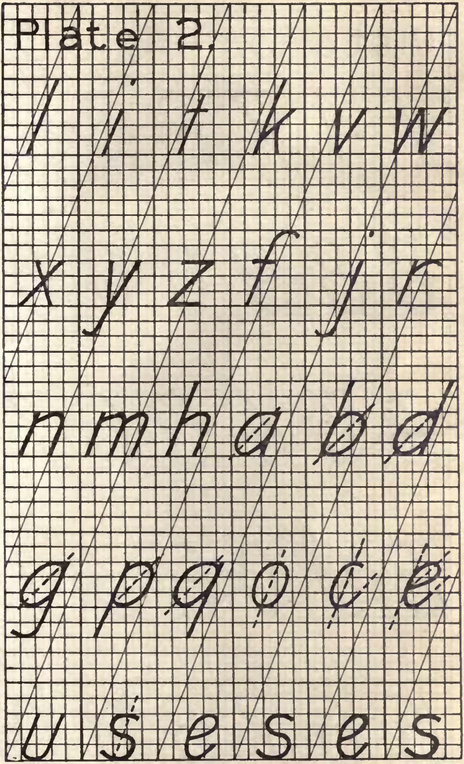


Plate 2.
l i t k v w
x y z f j r
n m h a b d
g p q o c e
u s e s e s

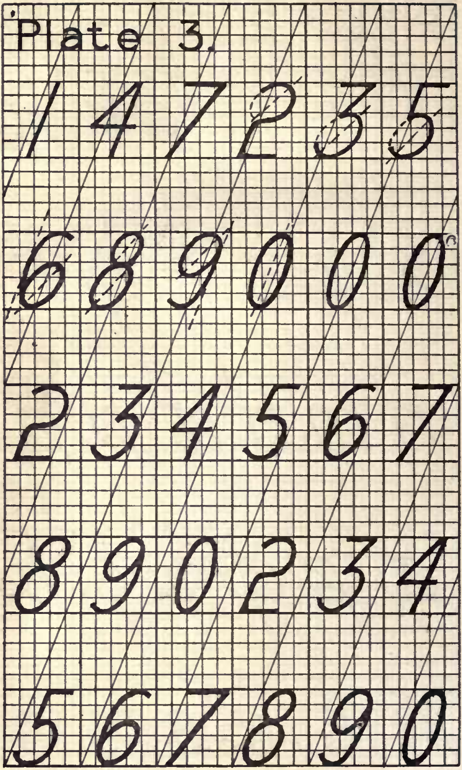


Plate 3.
147235
689000
234567
890234
567890

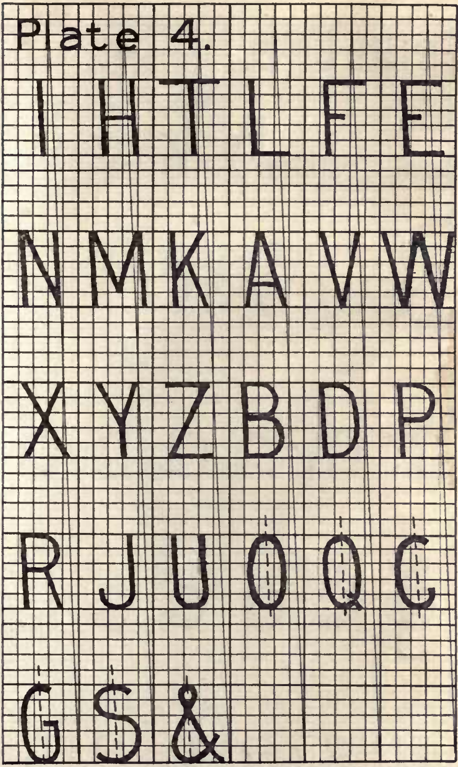


Plate 4.
I H T L F E
N M K A V W
X Y Z B D P
R J U O Q C
G S &

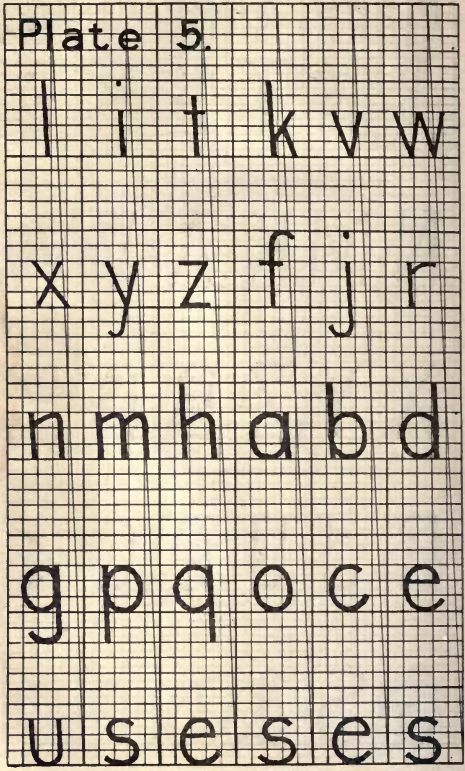


Plate 5.
l i t k v w
x y z f j r
n m h a b d
g p q o c e
u s e s e s

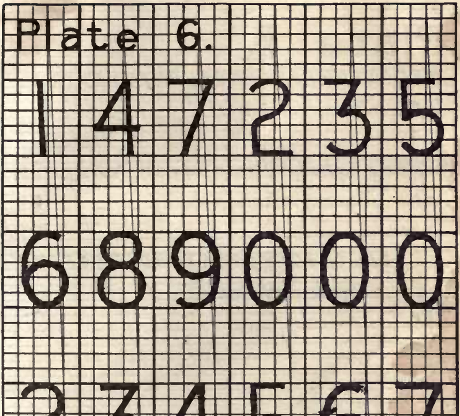


Plate 6.
147235
689000
234567
890234
567890

Plate 7.

ABCDEFGHIJKLMNOPQRSTUVWXYZ&

ABC (Repeat capitals.)

ABC " "

abcdefghijklmnopqrstuvwxyz

abc (Repeat lower case.)

abc " " "

1234567890123456789012345678

123 (Repeat numerals.)

123 " "

123 " "

123 " "

ABCDEFGHIJKLMNOPQRSTUVWXYZ&

ABC *(Repeat capitals.)*

ABC " "

abcdefghijklmnopqrstuvwxyz

abc *(Repeat lower case.)*

abc " " "

1234567890123456789012345678

123 *(Repeat numerals.)*

123 " "

123 " "

123 " "

UN